500.00 STRUCTURES - GENERAL (500)

Inspection

Inspection of the construction of structures is highly technical and demands the Inspector be completely informed on all phases of the operation. The inspector should be thoroughly familiar with the plans, specifications, and special provisions pertaining to a particular phase of construction prior to commencing construction operations. The inspector should be aware of the reasons behind each of the provisions listed in the specifications. These provisions have been developed through years of experience and research designed to obtain a quality product. A review and discussion of the provisions and the appropriate sections of this manual with the Contractor, Subcontractor and/or supplier will eliminate many misunderstandings. The first step in inspection is careful checking of the plans for errors. This should begin as soon as plans are available. Sub dimensions must be compared to overall dimensions and clearances and tolerances checked. Bearing elevations and anchor bolt locations must be carefully verified. Problems should be resolved with the Bridge Engineer through the Resident Engineer.

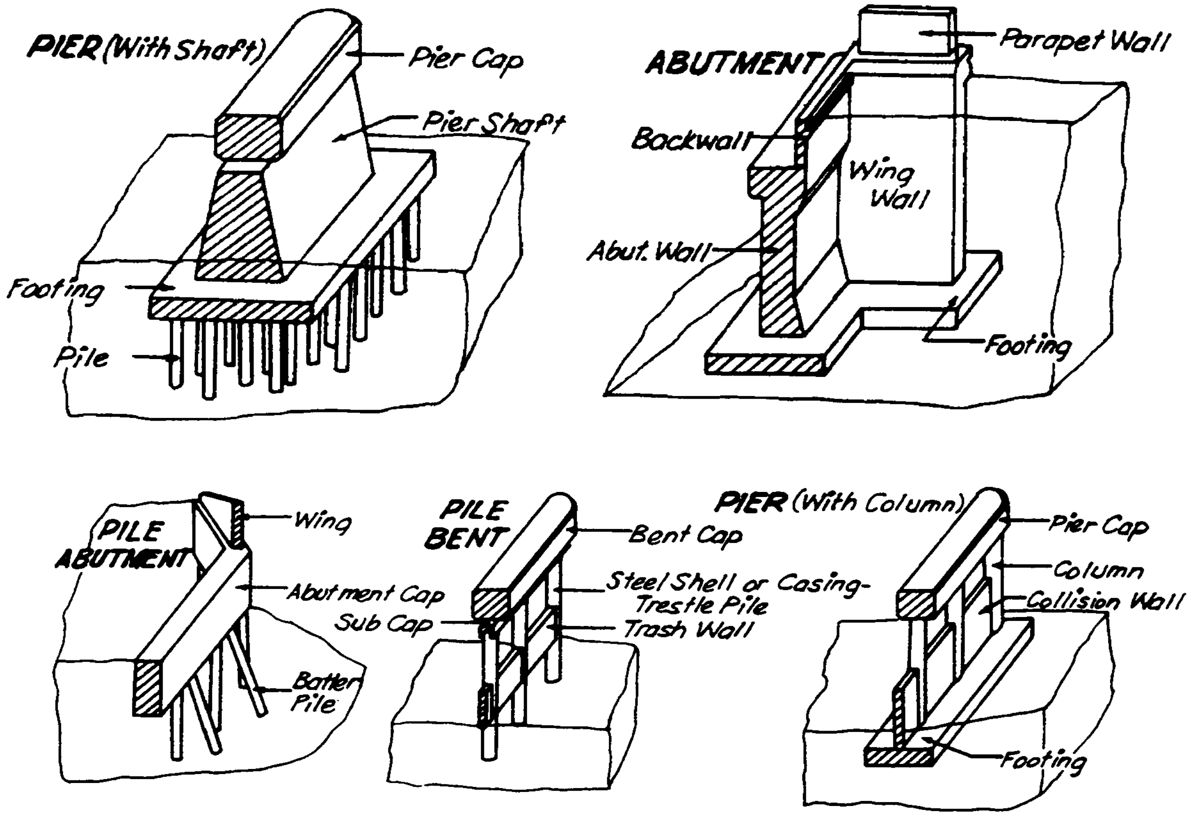
A check should be made from a distance to see that the item is in the correct place and proper position. Don't be so close to the work that you can't see the forest for the trees. Step back. Does the footing cover the piling? Does the skew angle fit conditions? Is there room for the other portions of the structure? Once concrete is placed or steel is bent or cut it is rather late to make changes.

Staking

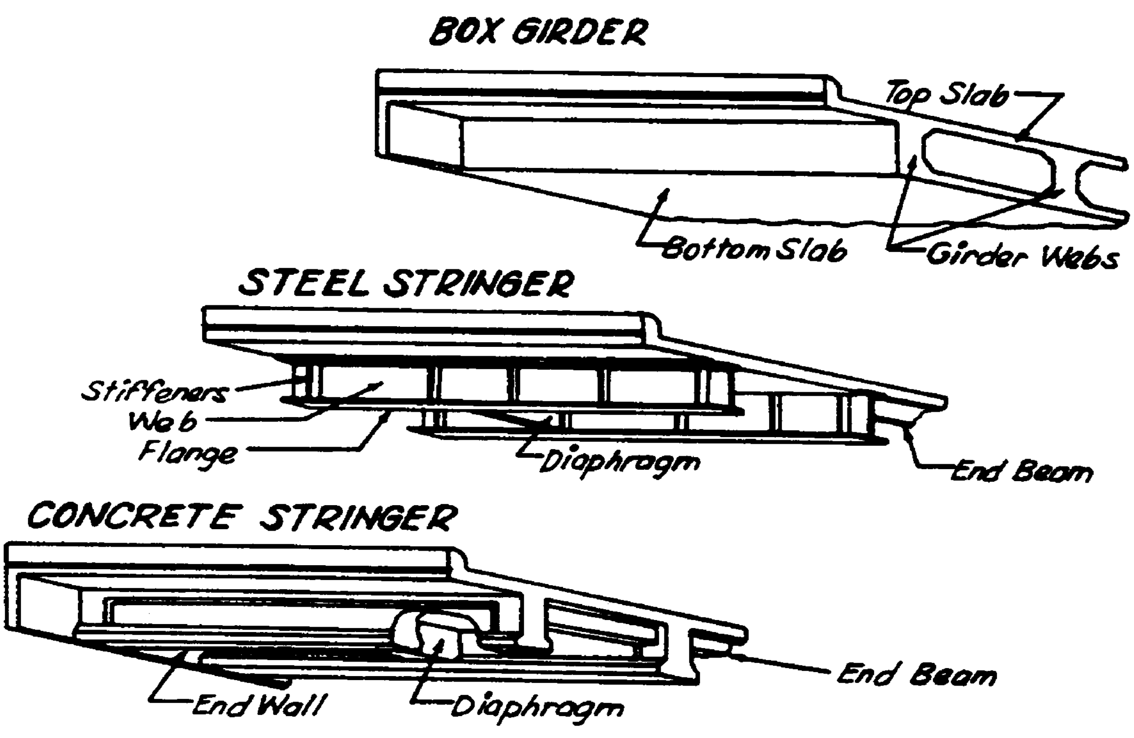
The responsibility for setting construction control stakes is outlined in the specifications. A question usually arises regarding the amount of staking that should be performed for the Contractor on structures. In some cases, the Engineer has elected to set all the necessary structure grades for the Contractor. This practice should be avoided for two reasons. First, by doing so, the Engineer has created a definite area of responsibility for errors which may result on the structure. Second, while performing this work the Engineer obligates personnel to duties which should be performed by the Contractor. It is, however, necessary to check grades and lines which have been set by the Contractor.

Adequate control staking for the structure will greatly assist the Contractor and provide a means of rapid checking by the Engineer's personnel. Control stakes should be located out of the area of operation of both the structure and roadway Contractors as much as possible. The contractor's personnel should be shown the location of these stakes and their purpose explained. Incomplete or vague marking may cause unnecessary delays or expensive corrections.

When setting grades, complete the circuit to a second bench mark thereby checking the elevation. A disturbed bench may not be discovered unless the grade is checked on a second bench.



SUB-STRUCTURE



SUPER STRUCTURE

501.00 STRUCTURES (501)

General

The design of a structure assumes an unyielding foundation, any settlement will affect the grade line and riding surface. Minor settlement can cause overstress of material, serious cracking and failure. It is of prime importance that the structure foundation be inspected to assure adequate bearing capacity; i.e., bearing values and foundation data shown on the plans should be compared to field conditions. Loose, disturbed material must be removed from the excavation and replaced with backfill in accordance with the specifications.

When excavation extends through stratified soils containing unsatisfactory materials, special probing or test holes may be required to check the material below the bottom of footing. This is especially true if layers above the bottom of footing do not conform to the test hole data. Always compare the actual material that is found against the boring information. Resolve any differences with the Construction Section.

Special care should be exercised in the placement of fills beneath structures. The use of granular fill material and the special control of compaction or compaction procedures may be required by the plans to attain the required density.

The material to be used behind abutments, retaining walls, etc., is to be free draining granular material. Refer to plans or special provisions for placement of this material and possible special drains.

Any shoring and cribbing required should be designed to allow sufficient space for placement of forms. Water must be channeled outside the forms for pumping. Underwater foundations requiring cofferdams should also provide adequate space for placing of forms and for handling water outside the footing. They should provide for a possible lowering of the footing elevation and be high enough to prevent overflowing of the cofferdam during high water. The Contractor should be made aware that any restriction in the channel due to forming may result in a raising of the water elevation, making it necessary to deepen or rechannel the flow to avoid flooding.

A log of material should be included in the daily diary, together with work accomplished and unusual occurrences or materials encountered. Photographs of conditions and the operations, identified as to time and location, are a valuable addition to the record.

When foundations are at a considerable depth below water, it may be necessary to provide a seal of concrete before attempting to de-water the cofferdam. This is done after the piling has been driven and/or the excavations to the final footing elevation has been completed. The purpose of the seal is to act as a counterbalance to the pressure created by the head of water on the outside of the cofferdam.

Documentation for Pay Quantity

Diaries are intended to provide a record of unusual or controversial happenings and to provide a detailed record of each phase of construction. They may be used in planning and organizing the work and for computation of quantities and may prove to be valuable references in connection with the performance or failure of some phase of work or may be used as evidence in court action to settle disputes between the Division of Highways and the Contractor.

In general, the Inspector's diary should include a record of all tests and measurements made and samples taken during the shift. Any communications with Contractor's personnel should be noted in detail and the notes should reflect compliance with the specifications. General observations should be made concerning weather conditions, water elevations, materials sources, and related information. Any incident affecting the progress of the work should be recorded (including cause, time, place, duration, number of men, and equipment made idle). There is always the possibility that a claim might arise because of a work stoppage and records should be made with this in mind.

The diary should be written completely before the end of each shift and nothing concerning the job should be considered too unimportant to include.

Field notes should not be copied but should be kept exactly as they are originally recorded.

Reports

None.

When unusual conditions or problems are encountered, a written report covering all the details should be submitted to the Bridge Engineer.

502.00 CONCRETE (502)

General

Unlike other materials used in highway construction, concrete is seldom removed and replaced. Therefore, it is essential that every precaution be exercised to insure that the initial placement is correct. To further assist the inspector in obtaining the product desired by the Department, the inspector must be thoroughly familiar with the Concrete Manual.

The Materials Section has set up a list of approved aggregate sources and batch plants (see Field Test Manual 16-865). These aggregate sources have been approved for aggregate quality.

Before new sources of aggregate can be used, they must first be tested and approved. High classes of concrete such as Class 35 or 40 (50 or 55) may not be obtainable from all "approved" aggregate sources.

The contractor must submit a concrete mix design for all classes of concrete. Each mix design, except Classes 10 and 15 (15 and 22), must be supported by test results indicating the design, under production conditions, will consistently provide average compressive strengths equal to or exceeding the minimum specified strength (concrete class times 100) multiplied by the appropriate overdesign factor. The overdesign factor shall be determined as described in Subsection 502.06 of the Standard Specifications. Recent state project concrete compressive strength test reports may be used to support mix designs in lieu of furnishing special samples and lab test reports. Providing the mix design is acceptable and the laboratory results indicate the mix will consistently meet the intended strengths, the Resident Engineer should write the contractor authorizing the contractor to use said mix design.

It should be pointed out that this is not an approval of the mix design only that supportive data demonstrates the mix design will consistently provide the strengths specified and that acceptance of the concrete is based on the 28-day concrete cylinder breaks (see sample letter, Exhibit 502-2).

Samples of cement, water, additives, sand, and coarse aggregate must be submitted at the start of the job and as required by the minimum test schedule.

Slump and air tests must be run on the first concrete delivered to verify that specifications are being met. A yield test must also be run on this concrete to determine primarily if the batched concrete contains specified minimum cement per cubic meter (CY). Concrete which over or under yields indicates that either the mix design is not being followed or adjustment in the design is necessary. Under yielding usually results in higher strengths and over-yielding in lower strengths. The ideal condition is when the yield is 100 percent.

Once a mix design has proven satisfactory, inconsistencies between loads can usually be traced to one of the following causes:

A. Failure to make the proper moisture content correction for the aggregate at the mixing plant. Changes in the stockpile moisture will certainly result in changes in the mix. Specifications require aggregates to be stockpiled or binned for drainage at least 12 hours before being batched.

B. Indiscriminately adding water to the mixture. The contractor may make minor adjustments to the mix proportions to improve workability as long as all basic concrete specifications are maintained. The minor adjustments should be approved by the engineer prior to implementation. If the contractor wants to add water on site, they shall have a batch ticket that shows how much water is allowed to be added to reach the maximum allowable water/cement ratio. Extremely hot weather and extended mixing time may stiffen the concrete mix. When necessary, the cement may have to be added at the job site to help concreting operations. Concrete temperature at time of placement must not exceed 27°C (80°F). It may be necessary to add shaved ice as part of the mixing water to keep the mix temperatures low enough.

C. Failure of the mixing or measuring equipment or the improper operation of this equipment. The specifications clearly outline the requirements that equipment must meet. It is essential that the inspector be aware of any shortcomings in the equipment and that the contractor takes corrective action before batching.

Falsework Inspection

In essence, the field engineer's rule as it pertains to falsework inspection is to ensure that the falsework, as it is constructed, complies with the following requirements:

"Falsework be designed and stamped by a licensed engineer registered in the State of Idaho. The drawings and computations must include design loadings and type of materials to be used. Falsework drawings and computations must be submitted to the engineer for review.

"The falsework shall be constructed to substantially conform to the falsework drawings.

"The materials used in the falsework construction shall be of a quality necessary to sustain the stresses required by the falsework design.

"The workmanship used in falsework construction shall be of such quality that the falsework will support the loads imposed on it without excessive settlement or take-up beyond that shown on the falsework drawings."

Experience shows that details give the most trouble. Falsework failures are seldom, if ever, a result of faulty design. Rather, failures almost always can be traced to the oversight of some minor detail.

Therefore, it is imperative that construction details be given special consideration, with particular attention to connections and details which contribute to the stability of the falsework system.

Falsework is usually erected on timber pads or sills set on the surface of the existing ground; although, occasionally, soil conditions are such as to require construction of concrete footings or driving of piles to ensure an adequate foundation for the falsework. In most cases, falsework will be composed of either steel or timber components, or members, or a combination of these two materials. The most frequently encountered combinations of falsework materials are:

Timber posts and caps with timber or steel stringers and timber joists. This type of construction is often referred to as "conventional" falsework.

Tubular steel pipe frame components assembled together to form towers. This system utilizes steel or timber stringers between towers with timber joists between the stringers.

Structural steel bents constructed from I or WF rolled shapes or from welded tube sections, supporting steel or timber stingers and timber joists. Steel bents are usually supported by and securely fastened to concrete footings or steel sills anchored to the pavement.

Timely inspection as falsework construction progresses is essential to effective Construction, and the contractor should be informed immediately when deficiencies are discovered. It does little good to wait until the falsework is erected to point out an inadequacy that should have been corrected on the day work began.

Falsework specifications require that construction of falsework may not begin until the engineer has checked the falsework drawings. This requirement is to be enforced on all projects, without exception. Note that leveling of ground will not be interpreted as "falsework construction" in administering this specification; but the placing of timber pads or the driving of falsework piles is "falsework construction" and will not be permitted in the absence of checked falsework drawings.

Prior to the start of construction of any falsework over or adjacent to the traveled way, the contractor should be reminded of his responsibility to provide for the safety of the public. The engineer has the responsibility and the authority to demand that all aspects of falsework construction, including workmanship and erection procedures, conform to the best engineering practice in any situation where public safety is involved. The engineer should not hesitate to require additional work or to direct or stop any construction procedures if such action is warranted to ensure public safety.

Conversations with the contractor concerning falsework construction should be recorded in the daily diary. If there are conditions which are critical and the contractor does not take corrective action, a written order should be given. The letter should state specifically what conditions need correcting, but should not dictate how. No predictions should be made. In no case should the falsework be loaded before satisfactory repair has been made.

In addition to routine falsework photographs, photographs of the falsework (including close-up photos of details) should be taken in all cases where the falsework has required extensive repair or upgrading in order to meet contract requirements.

The inspector should become familiar with the following phases of falsework inspection:

A. Foundations

Regardless of how well constructed the falsework may be, its ability to carry the imposed loads is no better than the foundation upon which it rests. Accordingly, falsework inspection should begin with an examination of the foundation material. Typically, falsework may be supported on soil, which may consist of native or imported material, or on rock, pavement or driven piles. Foundation problems most often occur when falsework is supported on soil; hence, most of this section is devoted to a discussion of soil and soil bearing values.

However, it should not be assumed that because falsework is supported on rock or piles, no inspection of the foundation is necessary.

Falsework special provisions, when provided, require falsework footings to be designed to carry the loads imposed upon them without exceeding the assumed soil bearing values or anticipated settlements. The soil bearing value assumed in the falsework design for both wet and dry conditions must be shown on the contractor's falsework plan. Actual values and soil condition must agree with the assumptions.

An inspection of the foundation materials should be made before the pads are set in place. The supporting capacity of the soil may be roughly estimated by probing with a piece of reinforcing bar. The bar may penetrate 0.3 m (1 ft.) or more in loose material, but will penetrate only 25-50 mm (1-2 in.) in compact material. The weight of an average sized man concentrated on the heel of one shoe exerts a force of approximately 145 kPa (1.5 tons/ft5). Consequently, if the material is firm to walk on without indentation, it should be capable of supporting a falsework loading of this magnitude. These simple field tests are indicators only, however, and should be used with judgment.

Falsework pads are often set on abutment fills, or on top of backfilled material around piers and columns. In these cases, care should be taken to ensure that the material is compacted as it is placed. This is particularly important in the case of backfill around piers or columns in stream channels or where traffic will be some distance away. Many so called falsework failures are actually attributed to excessive settlement of pads placed on improperly prepared soil.

Falsework pads should not be placed on the sloping surface of a cut or fill slope where they may be undermined or subjected to sliding downhill. Pads should be set on horizontal benches cut into firm material, with the pad set well back from the edge of the bench.

Many soils lose their supporting capacity when they become saturated. Adequate falsework construction provides for drainage to protect pads from being undermined or ponded in water.

It is not easy to determine the true bearing capacity of a given soil. Accordingly, the engineer should not hesitate to require a soil bearing test if he has any doubt whatsoever as to the ability of the foundation material to support the falsework load without settlement.

The Materials Section is available for consultation and advice as to the suitability of load tests in a given field situation, as well as interpretation of test results.

B. Materials

1. Timber

When inspecting falsework materials, the inspector should keep in mind that the primary responsibility is to prevent the use of materials which obviously do not meet the falsework design criteria. He is not expected to become a lumber grader.

In no case should the contractor be permitted to splice or block posts in bents adjacent to railroads or roadways; because it is imperative that they are stable at all times, including times when no appreciable dead load is acting. No bracing of any type should be allowed to be fastened to the temporary rail which protects the falsework adjacent to traffic.

2. Structural Steel

Steel beams and, particularly, salvaged members should be examined carefully for loss or change of section due to welding, rivet holes, or web openings. If the exact size or section of a used beam is not readily apparent, section properties usually can be determined with sufficient accuracy for verification of beam strength by field measurements.

Beams composed of short members which have been welded together to form a longer length should not be used for falsework at any critical location.

3. Manufactured Products

Manufactured products such as tubular steel shoring and overhang brackets are particularly vulnerable to damage by continual reuse. Fabricated units in which individual members are bent, twisted, or broken will show a substantial reduction in load-carrying capacity.

Steel shoring materials should be examined carefully prior to use. Shoring components should not be used if they are heavily rusted, bent, dented, rewelded, or have broken weldments or other defects. Connections, in particular, should be examined for evidence of cracked or broken welds. Miscellaneous components such as screw jack extensions, clamps, and adjusting pins should be inspected as well.

Manufacturer's ratings are based on the use of new material or used material in reasonably good condition. The determination as to whether a manufactured product is in "reasonably good condition" is highly subjective and requires experience and judgment.

Following is information on the more commonly used manufactured falsework products:

a. Standard Pipe-Frame Shoring

Falsework shoring composed of tubular steel members has gained wide acceptance during the past decade. This shoring system consists of end frames of various types which are erected in pairs and held rigidly together pin connected diagonal cross-braces. The pairs of frames may be stacked one above another to form towers, each tower being 1.2 m (4 ft.) wide (which is the frame width) and 2.4 or 3 meters (8 or 10 ft.) long. Frames are also available in 0.60 m (2 ft.) widths for special uses.

The base frames are 1.8 m (6 ft.) in height. Extension frames may be set at various positions to extend the base frame from 0.3 to 1.5 m (1 to 5 ft.). Minor vertical adjustments are made with screw jacks located at the top and bottom of the tower.

Two types of frames are in general use: The ladder type in which the frames have horizontal struts between the vertical legs and the cross-frame type in which lateral stability is provided by cross-bracing between the legs.

b. Deck Overhand Brackets

Several types of steel jacks or brackets especially designed to support cantilevered deck overhangs are available commercially. The manufacturer's recommended safe working loads should be followed. If a particular jack or bracket cannot be identified, a test load should be required.

The special provisions require falsework and forms to be so constructed that loads will be applied to the web of steel girders within 150 mm (6 in.) of a flange or stiffener. The loads must be distributed so as to prevent local distortion of the web. In addition, temporary struts and ties must be provided as necessary to resist lateral loads applied to girder flanges and to prevent appreciable relative vertical movement between the edge of deck form and the adjacent steel girder.

Lateral loads applied to girder flanges will produce an overturning moment in the girder. To prevent possible overstressing of the permanent end and intermediate diaphragm connections, the temporary struts and ties required by the specifications must be designed to resist the full overturning moment.

c. Beam Hangers

Beam hangers are basically hardware items which are placed transversely across the top flange of a beam or girder. Steel rods or bolts, which are inserted into threaded wire loops at the hanger ends, hang vertically and support the deck slab falsework.

Manufacturer's catalog data should be consulted to determine the safe working loads. Note that some manufacturers list total hanger capacity whereas others list values for one bolt or rod.

Unbalanced loading (i.e., loading only one side of the hanger) will materially reduce the load-carrying capacity of the hanger unless it is designed to be loaded on one side at a time, or unless special measures are taken to hold the hanger in place.

Beam hangers must not be welded to the top flange of a steel girder or to prestressed girder stirrups. Welding to shear connectors or studs is permissible, however, if approved by the engineer.

d. Steel Joist Assemblies

Joist assemblies, common building construction, are being used more and more frequently in bridge falsework.

Joist assemblies are essentially steel beams which can be adjusted to provide a wide range of span lengths. Manufacturer's catalog data should be consulted to determine the safe load-carrying capacity.

When joist assemblies are used to support deck slabs between girders, design load deflection is limited to the maximum deflection recommended by the manufacturer, which may exceed 1/270 of the span.

C. Workmanship

1. General

Workmanship should be of such quality that the falsework will support the loads imposed without excessive settlement or take-up beyond that shown on the falsework drawings.

Poor workmanship, particularly in such details as wedges, fasteners, bracing, jack extensions and the like, has been responsible for more falsework failures than inadequate design or overstressed materials. Accordingly, construction details should receive the field engineer's closest attention.

2. Timber Construction

Timber posts should be wedged at either the top or bottom for grade adjustment, but not at both locations. Large posts may require two sets of wedges to reduce compression stress perpendicular to the grain.

Blocking and wedging should be kept to a minimum. It is poor practice to extend a short post by piling up blocks and wedges. Wedges should be placed with a surfaced side next to a rough-cut side rather than two surfaced sides together.

Full bearing should be obtained between all members in contact. Deficiencies in this respect may be improved by feather wedging with a single shingle. Joints requiring more than a single shingle should be recut.

When wood shores are butt spliced, the splice shall be made with square joints adequately secured on all four sides with not less than 50 mm (2 in.) materials or 16 mm (5/8 in.) plywood of the same width as the post. The scab must extend 0.60 m (2 ft.) beyond the joint. Good practice limits splices to one per post.

The following workmanship checklist is included as a guide to points which may require special consideration:

a. The size and spacing of falsework members must agree with details shown on the falsework drawings.

b. Posts must be plumb and erected from level and even surfaces.

c. Blocking and wedging should be kept to a minimum. Too much blocking or too many wedges leads to instability.

d. Diagonal bracing, including connections, must agree with details shown on the falsework drawings.

e. Diagonal bracing should be inspected after the falsework has been adjusted to grade. Connections must be securely fastened (retighten if necessary to ensure their effectiveness in resisting horizontal forces).

f. The ends of spliced posts must be cut square, and scabs nailed securely on all four sides.

g. Full bearing should be provided at all contact surfaces.

h. Posts should be centered over the falsework pad or sill to ensure uniform soil load distribution.

i. Falsework pads must be uniformly supported by the foundation material.

j. Permanently deflected stringers should be placed with the crown turned upward.

k. The method of adjustment should be such that the falsework may be readily adjusted to grade.

l. Jacks used for adjustment should be plumb and not overextended.

m. Abutting edges of soffit plywood should be set parallel to the joists, and continuously supported on a common joist.

n. A sufficient number of telltales should be installed to accurately determine the amount of joint takeup and settlement. Telltales should be attached to the joists and as near as possible to the supporting post or bent.

3. Steel Shoring

The following inspection checklist is based on information in the "Recommended Standard Safety Code for Vertical Shoring" issued by the National Scaffolding and Shoring Institute. This checklist may be used as a guide by field engineers when inspecting falsework constructed of welded tubular steel shoring.

a. Shoring components should be inspected prior to erection. Shoring, including accessories, which is heavily rusted, bent, dented, or rewelded or which, if otherwise defective, shall not be used.

b. A base plate, shore head, extension device or adjustment screw shall be used at the top and bottom of each leg of every tower.

c. All base plates, shore heads, extension devices, and adjustment screws shall be in firm contact with the footing at the bottom and the cap or stringer at the top and shall be snug against the legs of the tower.

d. Shoring components should fit together evenly without any gap between the lower end of one unit and the upper end of the other unit. Any component which cannot be brought into proper contact with the component into or onto which it is intended to fit shall be removed and replaced.

e. Eccentric loads on shore heads and similar members shall be avoided.

f. All locking devices on frames and braces shall be in good working order, coupling pins shall align the frame or panel legs, pivoted cross braces shall have the center pivot in place, and all bracing components shall be in a condition similar to that of original manufacture.

g. Shoring shall be plumb in both directions. The maximum deviation from true vertical shall not exceed 3 mm (1/8 in.) in 1 m (3 ft.). If this deviation is exceeded, the shoring shall not be loaded until it is readjusted within this limit.

D. Field Changes

Some judgment will be required to determine whether the falsework as it is being constructed "substantially" conforms to the drawings.

As a matter of policy, the following changes will be considered substantial and must be shown on revised falsework drawings regardless of other considerations.

1. A change in the size or spacing of any main load carrying member.
2. A change in the method of providing lateral or longitudinal stability.
3. Any change, however minor, which affects the falsework to be constructed over or adjacent to a traffic opening.

E. Inspection During Concrete Placement

As concrete is being placed, the falsework should be inspected at frequent intervals for evidence of overstressing. In particular, look for the following indications of incipient failure:

1. Excessive compression at the tops and bottoms of posts and under the ends of stringers.
2. Excessive bending of stringers or shores.
3. Tilting of joists or stringers.
4. Pulling of nails in lateral bracing; movement or deflection of braces.
5. Excessive settlement of telltales.
6. Rotation of any member because of eccentric or cantilever loading conditions.

If, during concreting, any member deflects unduly or shows evidence of distress, such as splintering on the bottom of stringers, crushing of joints or wedges, etc., concreting should be stopped and the affected area strengthened by the addition or replacement of falsework members.

One important and often overlooked point is the effect of curing water on falsework foundations. Some means must be provided to prevent curing water from reaching and soaking the foundation material beneath the falsework bearing pads.

Forms

A. General

The Standard Specifications require all forms to be designed and to have the seal and signature of a registered engineer in the State of Idaho unless otherwise approved by the engineer. Should the engineer waive this requirement, he should be thoroughly satisfied that the forms are adequate.

Extreme pressures are applied to forms by the concrete and when vibration is done on the concrete, the concrete becomes fluid which increases the pressures even more. There should be adequate engineering in every forming system to prevent failures during concrete placement.

Many forms are commercially manufactured. Upon request, most manufacturers, will supply all design and allowable loading data for their systems. From this data, the engineer can usually analyze the forming system for structural adequacy. If the forms are a contractor built system, the engineer should determine that the system is capable of performing as intended. If there is any question do not hesitate to enforce the requirements that the forming system be designed and approved by a licensed engineer.

B. Deck Forms

Deck forms should also be approved as above. The most critical point for deck forms is the spacing of span girders, hangers, and overhang braces. Check to assure that proposed loads do not exceed the maximum design load of these components. It should also be pointed out that the forms must not exceed the maximum allowable deflection.

When the forms are approved, the contractor may proceed with forming of the deck. Care should be taken to insure tight fitting forms. Mortar running through holes in the forms can cause visual damage and in some cases structural damage to the concrete.

The engineer should calculate all deck grades especially for bridges on curves or variations in width which require some special assistance from the Bridge Design Section.

Be sure to adjust all forms to grade before the steel placement begins, then check the grade after the steel has been placed.

Before the concrete placement, the engineer should inspect the forms both from the top and from the underside to insure all the elements of the system have been properly placed and that no deficiencies exist. Final grade checks should be made on a "dry run" measuring down from the screed of the finishing machine to the mats of the reinforcing steel and to the deck forms.

C. Permanent Metal Concrete Forms

The Standard Specifications describe the attachment of permanent metal concrete form supports to the flanges of stringers and girders by permissible welds, bolts, clips, or other approved means. The description goes on to say, "However, welding of form supports to flanges of steel not considered weldable and to those portions of a flange subject to tensile stresses (areas where intermediate stiffeners are welded to bottom flange and are gapped at top flange) shall not be permitted."

The reference to intermediate stiffeners has caused some confusion which resulted in stringers without intermediate stiffeners receiving welds on the top flange in tensile areas.

The intent of this specification is to permit no welding to girder or stringer flanges subject to tensile stress. This would also apply to deck overhang supports which have been welded to the girder flange.

Intermittent fillet welds have been used to attach the form support angle to the girder or stringer flange in compression areas which is permissible. Since this seems to be the most economical method, the contractor will probably continue to weld directly to the flange in the compression areas and use straps, clips, or some other method in the tension areas.

The approved shop drawings should show the tension areas over supports where welding will not be used. If the shop drawings do not show this, request the information from the Bridge Engineer for all girders and stringers which are subject to tension before the contractor begins attaching the forms.

It is very important that welding is avoided in the tension areas since arc strikes (which cause gouging) and weld metal deposits (which cause an abrupt change in cross section), both result in areas of stress concentration. These stress concentrations are considered extremely detrimental to the fatigue strength of the girder or stringer as it flexes through many cycles of loading. The stress concentrations are the first places to develop fatigue cracks. An increase in section due to a weld can therefore have a similar effect to a decrease in section such as a piece of steel which has been cut or notched will break at that weakened location after a number of repetitious bends. Any welding which has been found in tensile stress areas of girder or stinger flanges must be removed by grinding the weld flush with the original flange surface. Any reduction of the flange cross section due to cutting or gouging must be avoided during this corrective work.

Concrete Placement

Prior to each large deck placement, a meeting with the contractor's foreman should be held to go over all aspects of the placement. The total number of men available and in particular the finishers and finishing equipment should be adequate for the size of placement.

Placement crews must work over the deck area during the placement operation. The inspector must be particularly watchful to see that reinforcing steel and forms remain in their intended location. It is very important that the deck finishing machine be operated over the full length of the deck segment before concrete placement begins in order to check cover on reinforcement and any possible screed rail deflection. All necessary corrections shall be made before the placement is started.

Equipment breakdowns and power failures sometimes occur during concrete placement. Therefore, the inspector should satisfy himself that an alternate placement procedure can be implemented and that certain items of standby equipment are available. An extra vibrator has saved the day on many projects.

Sampling and Testing

It is very important to insure that the ends of concrete cylinders are smooth. It has been proven by tests that rough irregular cylinder ends cause a reduction in compressive strength up to ten percent. The reductions appears to become greater as the compressive strength increases. There is no substitute for careful workmanship in preparing concrete cylinders.

The inspector is cautioned against poor practices resulting in irregular ends. Two of the most common are as follows:

A. Denting the bottom of the can with a tamping rod. This can be prevented by placing the can on a firm foundation.

B. Improper finishing of top. Either too much or too little concrete results in an unsatisfactory surface. Too little concrete is difficult to trowel finish properly. Too much material, if allowed to come in contact with the can lid, can result in an irregular or convex surface depending on the lid or a nonparallel surface if the lid is placed improperly.

With the can on a level surface, trowel finish the cylinder flush with the top of the can. Lightly place the can lid on the can (overnight), if possible, until the concrete is partially set. Then place the lid on firmly. Sealing of the lid too soon results in the concrete sticking to one side of the lid but not the other giving a nonparallel surface.

Seal Concrete

Seal concrete calls for extra cement to make up for losses during underwater placement. Special care must be exercised in the placement of concrete below the water surface to keep agitation to a minimum. Bottom dump buckets may be permitted in shallow water. When placing seal concrete with a bucket, it must meet the same general conditions as outlined for a tremie. It must be watertight, the outlet buried in the concrete, and no washing of the concrete shall occur.

Seal concrete is placed with a higher slump so it will flow out of the tremie or bucket and into final position with little working. The higher slump also aids in preventing foreign water from entering the concrete.

Care should be exercised to assure that the required depth of seal is obtained over the entire area. The excavation should be checked for high areas before the seal is placed and the surface of the placed seal should be checked for irregularities.

After the seal has obtained the required strength to withstand the hydrostatic pressure, the cofferdam may be dewatered. During this operation, the flow of water through the joints in the sheeting tends to seal with solids moving with the in flow. Slow pumping provides more time for this sealing to take place.

When the cofferdam is dewatered, the surface of the seal should be trenched to a sump area for pumping, piling cut to the required elevation and spacing checked. The footing is then formed and construction proceeds in a normal manner.

Curing

Proper curing is of major importance. The specifications require that all concrete surfaces be kept completely and continuously moist until a curing method, depending on the type of placement, is applied.

Acceptable methods of curing are given in Subsection 502.16 of the Standard Specifications.

High temperatures, low humidity, and windy conditions have an adverse effect on curing of concrete surfaces. Each of these conditions, or a combination, will cause shrinkage cracks in the surface of the concrete unless preventative measures are taken. The chart at the end of this section, Exhibit 502-1, shows how to arrive at an evaporation rate. An evaporation rate greater than 1 kg/m5/hr (0.2 lb/ft5/hr) will indicate potential problems; and some type of corrective procedures should be considered to change the placement operation. This may require placement at night or early morning hours when the temperatures are lower and perhaps less windy conditions.

Prior to deck placements a hygrometer and a wind meter should be obtained from the District Materials Section so that the rate of evaporation can be determined during placement. District Materials also has available literature for the Prevention of Plastic Cracking in Concrete.

Acceptance of Concrete

The following procedure shall be used regarding acceptance of concrete on projects except for concrete handled by certification:

A. Concrete acceptance is based on supplied concrete meeting the minimum requirements specified in Subsection 502.02 of the Standard Specifications; classification and the results of the 28 day compressive strength tests. Concrete failing to meet the intended strength by meeting the allowable strength will be subject to a penalty. Concrete not meeting the allowable strength will be removed at the contractor’s expense. Plastic concrete not meeting the requirements of Subsection 502.02 of the Standard Specifications should be rejected prior to placement.

   The allowable compressive strength "F" is the actual design stress at which the structure was designed plus a factor of safety as provided for by AASHTO standards. The allowable compressive strength may vary in each structural component. Therefore this value should be obtained from the bridge designer. Should there be no determination for the value of "F" as determined by the designer, the engineer should use a minimum value for "F" of 85% of the intended strength as a basis for acceptance or rejection of the concrete and applying penalties to the concrete.

B. Concrete which has been allowed to be placed but which has subsequently been damaged through neglect by the contractor in not following specifications will be removed and replaced at the contractor's expense if the damage is intolerable, or otherwise left in place with and appropriate penalty (Subsection 105.02 of the Standard Specifications).

Documentation for Pay Quatities

The quantities of concrete shall be calculated by the Concrete Inspector before construction begins. If the total quantity for each major structure is within one cubic meter (CY) of the plan quantity, no additional checking is necessary. If the difference is over a cubic meter (CY), the calculations should be rechecked in the residency. If there is a great difference, the figures may be submitted to the Bridge Section for checking. These computations and checks should be included as part of the project records and generally will be the source document for final pay quantity of these items. Minor structures should check within 0.1 of a cubic meter (CY).

The aforementioned calculations should show the quantity to be paid for in each portion of the structure. Payment is based on plan dimensions except where a change in the plan dimensions was required in the field.

The Resident Office Manager enters the quantity of each item as reported from the field in the field ledger. The office manager may be required to compute or check the quantities.

The inspector may inform the contractor of the concrete quantities they will receive payment for; but, under no circumstances, should the inspector inform the contractor of the amount of concrete to be ordered. This responsibility must remain with the contractor.

If the calculated pay quantities vary considerably from the amount of concrete ordered or batched, the inspector should determine the reason for the variation. Large area placements such as decks will readily consume additional concrete with no visible indication. Any wasted concrete should be so noted and the quantity estimated. By keeping track of the variations throughout the job, the inspector may easily account for the contractor's purchased amount as opposed to the amount he is paid. On large projects, the waste can be considerable.

The diary shall be used to verify the activity, date and location of the work and measurements.

Quantities for concrete shall be computed and reported to the nearest one-hundredth (.01) of a cubic m (CY). Round off to the nearest 0.1 cubic meter (CY) on the estimates.

Stringers shall be reported and paid to plan dimensions. Estimates should be rounded to the nearest 0.1 m (LF).

Reports

A Concrete Delivery Ticket, DH-70, is to be completed for each truck load of concrete (see Concrete Manual for example).

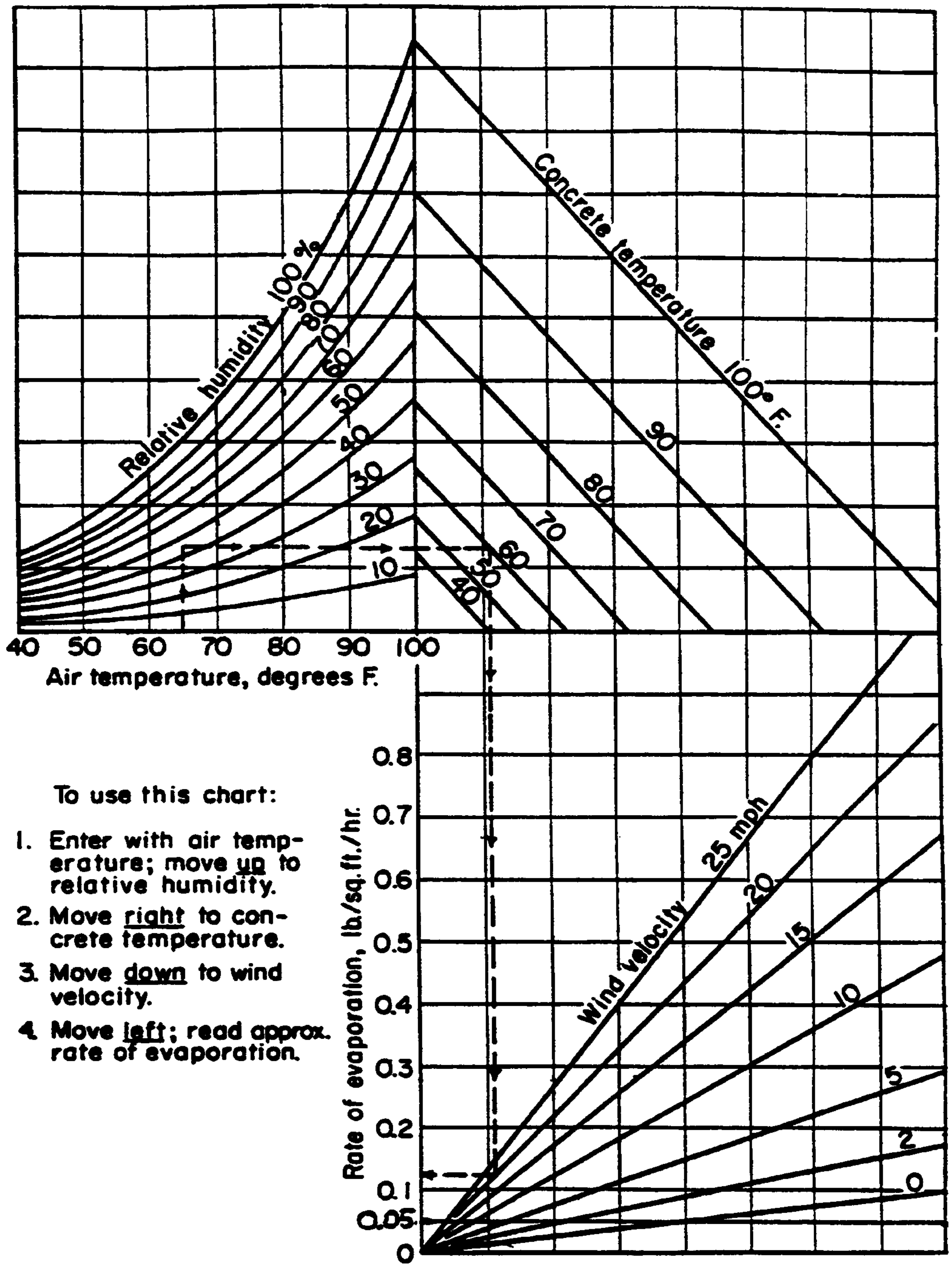


Fig. 58. Effect of concrete and air temperatures, relative humidity, and wind velocity on the rate of evaporation of surface moisture from concrete.

Portland Cement Association 1968.

Go-Get-Em Construction
P O Box 00
Anywhere, ID 83701

Gentlemen:

Your mix design number _____ and supportive data for concrete class __________ have been reviewed. The data indicates concrete produced using this mix design (should/does not) meet specifications.

Data for your plant indicates a coefficient of variation of _____% is appropriate for this mix. Application of the _____% overdesign factor results in a required basic mix strength of _____ psi. Based on test data, (laboratory tests/production tests on a minimum of 30 sets of test cylinders) submitted for this mix, the average mix strength is _____ psi, which (is/is not) satisfactory.

Specifications require the mix design to have sufficient cement so the minimum cement factor will be met at maximum allowable air content. The proposed mix (is/is not) satisfactory in this respect.

It should be noted that acceptance of concrete is based on field compliance with all specification requirements.

(If mix design does not comply, request contractor to correct non-applicable details and resubmit.)

(Add any other comments needed for project conditions.)

Sincerely,

L. M. Enno
Resident Engineer

cc: ready mix subcontractor
bcc: CAS, DE, ADE, DMC (w/copy of strength data), Matls (w/copy of strength data)

503.00 METAL REINFORCEMENT

General

On every set of bridge plans there is a diagram with notations describing the requirement for rod hooks and bends. A "hook" refers to the 90E, 135E or 180E bending of a reinforcing bar. A "bend" refers to any other degree of bend. Reference should be made to this diagram when inspecting metal reinforcement.

Following is a list of the types and purposes for which metal reinforcement is used.

A. Longitudinal Bars - Reinforcing steel bars which run length-wise of a member.

B. Transverse Bars - Reinforcing steel bars which run across the width of a member.

C. Stirrups - "U" or "W" shaped bars placed in a vertical plane used to resist shear in structural members.

D. Tie Bars - Act to resist stress and hold other stress bars and structural members in position.

E. Dowels - Short bars extending from one member into another to transfer stresses from one member into another.

F. Spiral Reinforcement - Spiraled hoops of reinforcing steel used on certain types of columns.

G. Temperature and Shrinkage Steel - Usually 10 M (#4) bars at 450 mm (18 in.) spacing placed in lieu of stress bars to resist tension cracking due to curing and temperature shrinkage.

Normal, still air cooling is recommended when bars are heated for bending. Quenching or rapid cooling of bars heated cherry red will produce hardening and brittleness in the steel and must not be permitted.

Steel reinforcement shall be protected from abuse and damage. Steel which is to be stored at the job site for an extended period should be protected from the weather to prevent excessive rusting. If covers are used to protect the steel, be sure to provide ventilation to prevent trapping of moisture in the enclosure.

A slight rust coating on the bars has little effect on the strength or on the bond to the concrete; however, scale, paint, oil, grease, mud, curing compound, dried mortar, etc., adhering to the bars must be removed. Heavy rust pitting could materially reduce the cross sectional area of the bar and should be cause for rejection.

When bars arrive on the job, they will normally be bundled and tagged. Each bundle will include all bars of a particular bend or schedule occurring in a certain portion of the structure. Each bar or bundle will bear a tag indicating the "mark" of the bar or bars. Some bars, all of a certain "mark," will be coded by a dab of paint on the end. This is to facilitate placement and identification.

The following table of permissible variations from plan location or spacing shall be used as a guide in determining good construction practice for placement of reinforcing steel. Substantial conformance to these values will be required.

	Beams	**Slabs**	**Walls**	**Piers & Columns**
Clearance from forms**	"6 mm "(3 in.)	"6 mm "(3 in.)	"6 mm*** "(3 in.)	"6 mm "(3 in.)
Spacing between top & bottom bars	------	"6 mm "(3 in.)	------	------
Spacing between parallel bars*	"6 mm "(3 in.)	"25 mm "(1 in.)	"25 mm "(1 in.)	"25 mm "(1 in.)
Placement of bars length-wise & Location of bend points***	"25 mm "(1 in.)	"50 mm "(2 in.)	------	------
Stirrup projection above top of beams	"13 mm "(2 in.)	------	------	------
Stirrup and hoop spacing	"25 mm "(1 in.)	------	------	"25 mm "(1 in.)

*The number of bars in any 1.2 m (4 ft.) of distance shall equal the number called for on the plans.
**End cover shall not be less than 25 mm (1 in.).
***13 mm (2 in.) for walls more than 200 mm (8 in.) thick.

The inspector should check the re-bar upon arrival at the job site to see that the bending has been done in such a degree of accuracy that placement can be made within the above tolerances. Items involving several bends such as stirrups are difficult to bend at the plant and these dimensions must be exact if the rest of the placement is to be correct. If it becomes necessary to reject reinforcement due to improper bending, the time to do so is prior to placement.

Just prior to the start of the concrete placement, a final check must be made to insure that the re-bar is properly positioned and is held securely in place. Specifications require that the deck finishing machine be operated over the full length of bridge deck prior to concrete placement. This is the time to check the top steel for proper cover. The force exerted by concrete as it moves into final position can move individual bars, mats, or cages out of position very easily. Top layers of re-bar in bridge decks must be tied down per each 1.5 m5 (16 ft5) of deck area. Cages in walls must be securely attached to the forms, not freestanding or spaced with temporary blocks. Metal chairs bend quite easily when stepped on. They require constant checking during the placement.

Check to assure that no steel is displaced by runways, accidents, dumping concrete, etc. Due to the low slump deck concrete, it is conceivable that the reinforcement could be displaced upward during the necessary vibrating operation. Constant attention is required to detect and correct any displacement of reinforcement. Also check on other items such as anchor bolts, inserts, pipe sleeves, conduits, etc.

Metal reinforcement being shipped to project from suppliers may be precertified in accordance with instructions from the Materials Section.

If the re-bar is not precertified, samples shall be taken and the contractor advised that use of re-bar prior to receipt of test results is at his risk.

The welding of stressed reinforcing steel may be permitted if such welding conforms with AWS D12.1, "Recommended Practices for Welding Reinforcing Steel, Metal Inserts and Connections in Reinforced Concrete Construction." Tack welding of non-stressed reinforcing steel is permitted. The amount of tack welding should be held to a minimum.

In ordinary slab-and-girder construction, only the top longitudinal slab steel is non-stressed. In column construction, all longitudinal steel is stressed; ties are non-stressed. In ordinary reinforced concrete beams, girders, and pier and bent caps, all top and bottom longitudinal steel is stressed; stirrups are generally stressed; small longitudinal bars which are not a part of top or bottom reinforcement groups are generally non stressed. In precast, prestressed concrete girders, stirrups are generally stressed; longitudinal non-prestressed steel is generally non-stressed although there are exceptions. Steel in footings is generally stressed.

Steel in the outer faces of retaining walls, wing walls, and parapets is generally non-stressed. Vertical steel at the inner faces of parapets and the earth faces of retaining walls and wing walls is generally stressed; here again, there are exceptions. When a stressed reinforcing bar terminates with a hook, the extreme end of the bar at the hooked end may be considered non-stressed.

Exhibit 503-1 at the end of this section shows general information and identification markings on ASTM Standard bars.

Documentation for Pay Quantities

The quantities of metal reinforcement should be calculated by the Concrete Inspector before construction begins. If the total quantity for each item is within 45 kg (100 lbs) of the plan quantity, no additional checking is necessary. If the difference is greater, the calculations should be checked in the residency. The contractor and his supplier should be informed immediately of any errors discovered during this checking. The Bridge Section should be consulted if the error involved is in the size of a bar. These computations and checks should be included as part of the project records and will be the source document for the final pay quantity of these items.

The above mentioned calculations should show the quantity to be paid for each portion of the structure.

The diary shall be used to verify the activity, date, and location of work. It is good practice to take photographs of the steel placement for further documentation and project information.

Quantities for payment of metal reinforcement shall be computed to the nearest 0.1 kilogram (lb.) and rounded off to the nearest whole kilogram (lb.) for each structure.

Reports

None.

PHYSICAL REQUIREMENTS FOR 1975 STANDARD ASTM DEFORMED REINFORCING BARS(a)

Type of Steel and ASTM Specification No.	Size Nos Inclusive	Grade	Tensile Strength Min., psi	Yield (b) Min., psi
Billet Steel A615	#3-#11	40	70,000	40,000
	#3-#11 #14 #18	60	90,000	60,000
Rail Steel A616	#3-#11	50	80,000	50,000
	#3-#11	60	90,000	60,000
Axle Steel A617	#3-#11	40	70,000	40,000
	#3-#11	60	90,000	60,000
Low Alloy Steel A706	#3-#11 #14,#18	60	80,000 (c)	60,000

(a) For more detail refer to the 22nd edition of CRSI/WCRSI *Manual of Standard Practice*, May 1, 1976.

(b) Yield point or yield strength. See specifications.

(c) Actual tensile strength shall not be less than 1.25 times the actual yield strength (A706 only).

Other useful references and handbooks:

Design and Construction, CRCP, 1968—CRSI

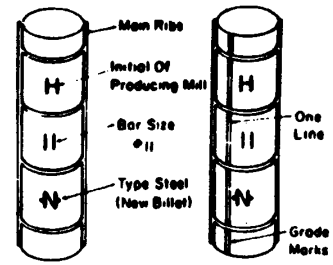
Reinforcing Bar Testing, 1974, ARBP-CRSI

Reinforcing Bar Splices, 3rd Edition, 1975—CRSI

Placing Reinforcing Bars, 3rd Edition, 1976—CRSI

IDENTIFICATION MARKS — ASTM STANDARD BARS

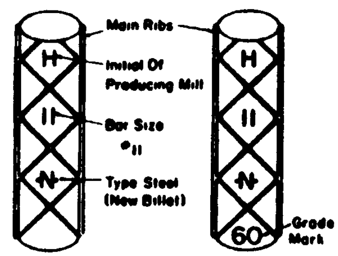
LINE SYSTEM — GRADE MARKS



Grade 40
Grade 50

Grade 60

NUMBER SYSTEM — GRADE MARKS



Grade 40
Grade 50

Grade 60

VARIATIONS: Bar identification marks may also be oriented to read horizontally (at 90° to those above).

Grade mark lines must be continued at least 5 deformation spaces.

Grade mark numbers may be placed within separate consecutive deformation spaces to read vertically or horizontally.

The ASTM specifications for billet steel, rail steel, axle steel and low alloy steel reinforcing bars (A615, A616, A617 and A706) require identification marks to be rolled into the surface of one side of the bar to denote the producer's mill designation, bar size, type of steel and, for Grade 60, a grade mark indicating yield strength.

Grade 40 and Grade 50 bars show only three marks (no grade mark) in the following order.

1st — Producing Mill (usually an initial)
2nd — Bar Size Number (#3 through #18)
3rd — Type Steel: N for New Billet
(S for Supplementary Requirements to A615 for #14 and #18)
A for Axle
I for Rail
W for Low Alloy

Grade 60 bars must also show a minimum yield designation grade mark of either the number 60 or one (1) grade mark line.

A grade mark line is smaller and between the two main ribs which are on opposite sides of all U.S. made bars. When a number grade mark is used, it is 4th in order.

ASTM STANDARD REINFORCING BARS

BAR SIZE	AREA SQ INCHES	WEIGHT POUNDS PER FT	DIAMETER INCHES
#3	.11	.376	.375
#4	.20	.668	.500
#5	.31	1.043	.625
#6	.44	1.502	.750
#7	.60	2.044	.875
#8	.79	2.670	1.000
#9	1.00	3.400	1.128
#10	1.27	4.303	1.270
#11	1.56	5.313	1.410
#14	2.25	7.650	1.693
#18	4.00	13.600	2.257

32 MISSOURI ROLLING MILL CORP. N Grade 40 only	38 OWEN ELECTRIC STEEL COMPANY N	50 TEXAS STEEL COMPANY N
32 MISSOURI ROLLING MILL CORP. I Grade 40 only	39 PACIFIC STATES STEEL CORP. N	51 U. S. STEEL CORPORATION N McDonald Mill
33 NEW JERSEY STEEL & STRUCTURAL CORP. N Bars #3 through #5 only	40 PENN-DIXIE STEEL CORP. Joliet Bar Mill Division N	51 U. S. STEEL CORPORATION N Birmingham Mill
33 NEW JERSEY STEEL & STRUCTURAL CORP. N Bars #6 through #11 only	41 REPUBLIC STEEL CORPORATION N	51 U. S. STEEL CORPORATION N Gary Mill
34 NORTH STAR STEEL COMPANY N Grade mark line on opposite side Bars #4 through #11, inclusive	42 ROANOKE ELECTRIC STEEL CORP. N	51 U. S. STEEL CORPORATION N Fairless Mill
34 NORTH STAR STEEL COMPANY N Grade mark line on opposite side Bars #14 and #18 only	43 SCHINDLER BROS. STEEL CO. N	51 U. S. STEEL CORPORATION N Lorain Mill
34 NORTH STAR STEEL COMPANY N Bars #8 through #18 (Patented) Marking system not in conformance with ASTM Specifications	44 SOULE'STEEL COMPANY N	51 U. S. STEEL CORPORATION N Torrance Mill
34 NORTH STAR STEEL COMPANY N Bars #3-#11 only Grade mark line on opposite side Wilton, Iowa Mill	45 SOUTHERN ELECTRIC STEEL CO. Division of the Ceco Corporation N	52 WITTEMAN STEEL MILLS N
35 NORTHWEST STEEL ROLLING MILLS, INC. N Bars #3 through #9 only	46 SOUTHWEST STEEL ROLLING MILLS Div of Cascade Steel Rolling Mills Inc N	53 YOUNGSTOWN SHEET & TUBE CO. N Grade mark line on opposite side Youngstown Mill
35 NORTHWEST STEEL ROLLING MILLS, INC. N Bars #10 through #18 only	47 STEEL SERVICE COMPANY Division of Azcon Corporation N Bars #3 through #11 only	53 YOUNGSTOWN SHEET & TUBE CO. N Grade mark line on opposite side Chicago Mill
36 NORTHWESTERN STEEL & WIRE CO. N Grade mark line on opposite side	48 STRUCTURAL METALS, INC. N	ARBP CRSI Founded 1974 This guide prepared through the courtesy of ASSOCIATED REINFORCING BAR PRODUCERS—CRSI Membership in the Associated Reinforcing Bar Producers—CRSI is open to all producers of reinforcing steel
37 NUCOR CORPORATION N Bars #4 through #18 inclusive	49 TENNESSEE FORGING STEEL CORP. N	

504.00 STRUCTURAL METALS

General

The inspector on the project should become familiar with the outside appearance of acceptable fabrication, even though shop inspection is usually made by a testing laboratory. The size and tolerances of rolled shapes may be spot checked against the AISC Manual for Steel Construction to assure that a lighter weight shape of the same series has not inadvertently been delivered. Fabricated shapes must be inspected for cracked welds, loose or misshapen rivets, bent stiffeners and clip angles, bent flanges, torn or buckled plates, and other damage which might occur in shipping. A list of defects, which may be cause for rejection, is sometimes furnished by the agency which has inspected the shop fabrication. These items should be duly noted along with the corrective action taken. Material stored on the project should be protected from damage.

Before erection, each member should be identified for proper location in the structure. Members are seldom interchangeable, because of location of bolt holes, special bevels, cuts or hangers. A plan must be developed for proper erection sequence, including details of field connections which are usually omitted from the contract plans. Sufficient equipment with adequate capacity and reach is needed to erect the steel without accident to men or material. Long girders may require more attention than weight alone would indicate because of lack of stiffness. Severe damage and over stressing may result from careless handling and erection.

Bearing surfaces of the structural members and of the substructure must be inspected for cleanness and freedom from defect. Contact of bearing surfaces, alignment, clearances, and in place camber, are items that require checking as erection proceeds and field connections are made.

Field connections are normally restricted to bolts because of the difficulty and expense of properly inspecting welds in the field. Where field welded connections are permitted or specified, it may generally be said that a weld with a uniform appearance after slag has been removed is a satisfactory weld. Rough or slipshod appearance and defective welds often go together. This rule does not apply when the practice known as slugging (filling with unburned rod) has been resorted to for speed. This practice can be detected by watching while the weld is made or by x-ray. The Central Laboratory has an ultrasonic flaw detector which can check field welds and any apparent deficiencies such as laminations or inclusions in the members. Two Central Laboratory inspectors, trained in ultrasonic testing, are also available to assist in field inspections on short notice.

Field connections are often permanently bolted. Bolted joints gain much of their strength from friction between the surfaces held together. This friction depends on bolt tension, which is a primary importance.

Special high strength bolts are used so that proper tension may be applied without stretching or breaking the bolt. A copy of Section 2.10.20 of the Standard Specifications for Highway Bridges adopted by AASHTO covering connections using high strength bolts should be obtained and enforced. Bolts should be checked carefully for compliance with material specifications.

Proper bolt tension may be applied or by the "turn-of-the-nut method" described in Section 504.06 of the Standard Specifications.

Load indicator washers are something new on the market. If used correctly, they will give satisfactory results. The engineer must check the specifications to be sure this type of device is approved for the project and install them in accordance with the manufacturer's recommendations.

The field procedure for installing high tensile bolts is as follows:

"Flair-up holes with enough pins to maintain dimensions and plumbness of the structure. Pins are not to be removed until bolts in all other holes have been tightened.

"Install bolts in remaining holes.

"Tighten a pattern of bolts as would be done for riveting, being sure that the connected parts are properly fitted.

"Using a spud wrench, tighten the nut of each bolt not used for fit up to a snug position, then continue to tighten it by turn-of-nut as specified, depending on grip length. 'Snug' is indicated by one-man maximum torque using a spud wrench.

"Replace pins with bolts and tighten as above.

"Back off the nut on each fit-up bolt (these bolts have previously been marked) then tension as per AASHTO turn-of-nut method.

"The bolt up crew should mark completed work with an identifying symbol."

Safety and safe practices are of paramount importance in all phases of structural steel work. The Inspector should become familiar with the Idaho Code of Minimum Safety Standards and Practices and be constantly alert to hazards.

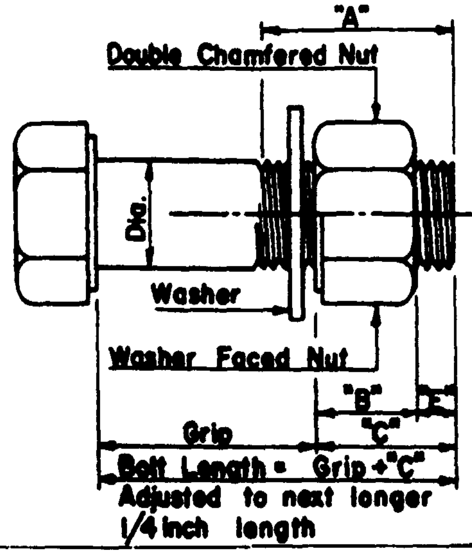
Torque wrench and calibrated impact wrench methods shall not be used for tightening of bolted connections.

Documentation for Pay Quantities

See Section 504.10 of the Standard Specifications.

Reports

None.



HIGH STRENGTH BOLTS
ASTM-A-325 AASHO-M-164
TYPE I, II, III

"A" Thread Length

"B" Nut Height

"C" Amount to add for Bolt Length

"E" Approximate Stick Thru

"T" Min. Tightened Bolt Tension in Lbs.

"S" Approximate Torque in Ft/Lb for tighened bolt tension

Dia.	"A"	"B"	"C"	"E"	"T"	"S"*
1/2"	1"	31/64"	11/16"	3/16"-7/16"	12,050	102
5/8"	1 1/4"	39/64"	7/8"	1/4"-1/2"	19,200	200
3/4"	1 3/8"	47/64"	1"	1/4"-1/2"	28,400	360
7/8"	1 1/2"	55/64"	1 1/8"	1/4"-1/2"	39,250	530
1"	1 3/4"	63/64"	1 1/4"	1/4"-1/2"	51,500	790
1 1/8"	2"	1 7/64"	1 1/2"	3/8"-5/8"	56,450	1060
1 1/4"	2"	1 7/32"	1 5/8"	3/8"-5/8"	71,700	1500
1 3/8"	2 1/4"	1 11/32"	1 3/4"	3/8"-5/8"	85,450	1970
1 1/2"	2 1/4"	1 15/32"	1 7/8"	3/8"-5/8"	104,000	2600

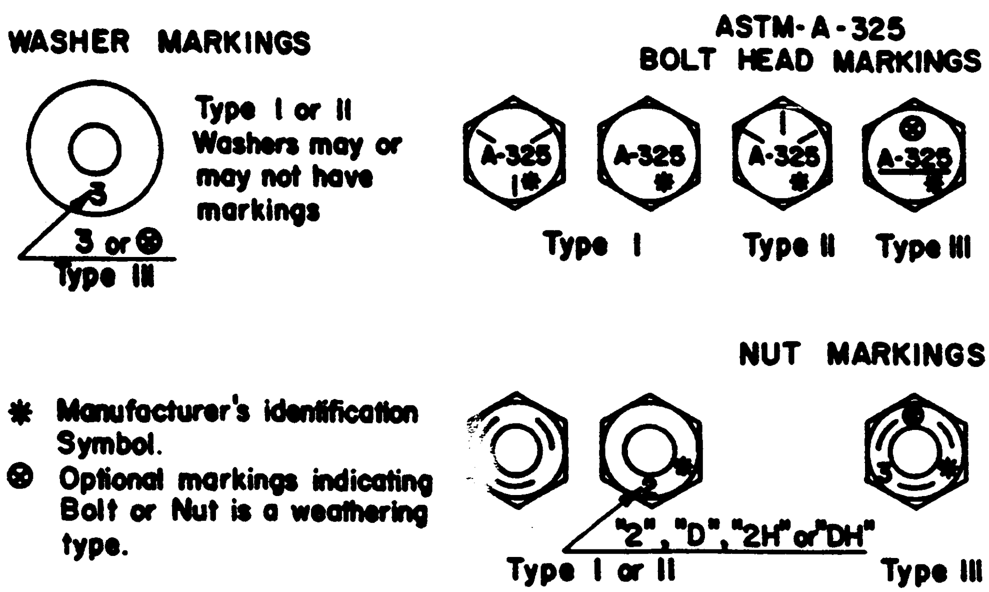
*Torque must be determined by bolt tension calibrator in field daily.

Turn-of-nut methods.
Bolt to be snug tight (full effort of man using ordinary spud wrench) then tightened additional as shown below:

TABLE 4—NUT ROTATION* FROM SNUG-TIGHT CONDITION

[illegible]

Bolt Length (as measured from underside of head to extreme end of point)	Disposition of Outer Faces of Bolted Parts		
	Both faces normal to bolt axis	One face normal to bolt axis and other face sloped not more than 1:20 (bevel washer not used)	Both faces sloped not more than 1:20 from normal to bolt axis (bevel washers not used)
Up to and including 4 diameters	1/3 turn	1/2 turn	2/3 turn
Over 4 diameters but not exceeding 8 diameters	1/2 turn	2/3 turn	5/6 turn
Over 8 diameters but not exceeding 12 diameters	2/3 turn	5/6 turn	1 turn



Type I - Medium carbon steel - sized ½" to 1½"D. Furnished unless noted.
Type II - Low carbon steel - sizes ½" to 1"D.
Type III - Weathering steel equal to ASTM-A-588 - A-242 - sizes ½" to 1½"D.
Bolt substitution (if approved). Type II and III for Type I, Type III for Type II.

A washer is required under the turning element (bolt head or nut).
Bolt holes may be punched, drilled, or reamed, 1/16" oversize, larger size and slotted holes may be approved by Engineer.
Joint surfaces shall be free of burrs, dirt, paint and foreign material. The light coating of oil on bolts and nuts as furnished by manufacturer need not be removed.
Bolts may be galvanized, but galvanized contact surfaces must be wire brushed (light brushed) or blasted (light brush off). An approved lubricant is required on bolt threads of galvanized bolts only. The nut should be free running on the bolt.
Tightening of bolts (snugging up and final tightening) in a joint should commence at the most rigidly fixed or stiffest point and progress toward the free edges.
When turn-of-nut method is used, location of turning element will be marked, before final torque, in order to provide the inspector with a visual means of checking the amount of rotation.

505.00 PILING (505)

General

A check of the contractor's pile driving equipment, accessories, and the piling is the first item on the agenda. Piling shall not be driven until the contractor has furnished the documents required in the Field Test Manual, Part II Quality Control, Item 505, for each type of piling to be used.

Piles shall be driven at locations as shown in the plans. Minimum pile spacings are generally 2.5 times the pile diameter. The inspector should check the plumbness and the location of each pile after it is installed. The tolerance for plumbness is one-half inch deviation from plumb for every foot of pile extending above the ground (0.04 ft./ft.). However, the pile top location, at cutoff level, should not deviate more than six (6) inches in any direction from the location shown in the plans. For steel H-piles, pile orientation is very important because the lateral resistance to bending of these piles depends on the flange direction. The inspector should check the orientation of H-piles to ensure that rotational deviation is not more than 30 degrees from that shown in the plans. Notify the Geotechnical Engineer at Central Materials Laboratory of any driven pile which does not meet the above criteria, as soon as possible, so that a determination may be made for any necessary corrective actions.

In plastic soil, a heaving may occur when adjacent piles are driven. For this reason, the inspector should check elevations on pile groups after they are driven. Any piles which have heaved shall be re-driven before the piling group will be accepted. The Design and Installation of Driven Pile Foundations, APF, recommends that piling heaving in excess of 6 mm (1/4 in.) be reseated.

It may be difficult to obtain the minimum penetration when driving piles through granular fills, very dense sand and gravel formations, boulders or certain silts. If refusal occurs before minimum penetration is obtained, the contractor may be required to try a larger size hammer, preboring the hole or jetting, depending on contract requirements.

Pile points or shoes may be needed to help piles penetrate hard soils or key into bedrock. Lists of approved points and shoes for steel piles are shown in Exhibits 505-7.1 and 505-7.2.

When preboring is employed, the bore hole should be smaller than the pile and stopped short of required penetration. Preboring should generally be stopped at least three feet or less above the estimated pile tip elevation as required by soil conditions. The piling must be driven to its final position. In very hard materials, such as bedrock, the bored hole is generally slightly larger than the pile and should be stopped at the design pile tip elevation. Pile is then driven to refusal. The annulus of the bored holes in rock shall be grouted to rock surface. The rest of the bored holes can be backfilled with grout or pea gravel, or approved materials.

When jetting, care must be exercised to ensure that the point of the jet is near the point of the pile. Considerable care must be taken when applying the jet to keep the advanced erosion ahead of the pile straight. The pile can easily be drawn out of position, as it usually tends to move toward the jet. It may be necessary to move the jet to the opposite side of the pile whenever lateral movement occurs.

The jet must be withdrawn before final penetration is reached and the final penetration secured by driving with the hammer. The three-feet-or-greater rule, as previously mentioned, applies to jetting also. Provisions shall be made to dispose of excess water through pipes or flumes so that adjacent fills or banks shall not be damaged. Dirty water will be handled and treated as required to prevent pollution of adjacent waterways.

Vibratory hammers can be used to drive piles but must be stopped at least 3 feet above the proposed pile tip elevation. Impact hammer then shall be used to drive piles to tip elevation or required pile bearing.

Steel Pipe Piling: The steel pipe piles shall be carefully checked for outside and inside diameter. Diameter and thickness of end closures also shall be checked. After driving, the pile should be checked for damage that may occur during driving. This can be done by either lowering a light to the bottom of the pipe or using a mirror to reflect the sunlight into the inside of the pile. Pipe pile should be covered, after driving, to keep out debris. Piles that contain water must be pumped or bailed out immediately prior to placing concrete.

H-Beam Piling: H-beams shall be handled in such a manner to prevent bending flanges, and supported when stacked for storage so they will not be damaged. H-piles shall be checked for square ends, straightness, and constant width between flanges throughout the length of the pile.

Precast (Prestressed) Concrete Piling: Precast concrete piles shall be checked for geometry, age at time of delivery, and strength of concrete. Proper handling and storage procedures must be followed. Condition of the piles must be satisfactory, such as no fissures, spalls, void pockets, aggregate segregation, etc. Shop drawings should be reviewed to determine the location of "lifting points."

Splicing:

Timber Piles: Contact the Contract Administration Section before permitting splices.

Metal Piles and Precast Concrete Piles: Splicing may be allowed upon approval of the splicing method by the Bridge Section.

Markings: All piles should be marked in increments sufficient to determine bearing capacity and penetration at all elevations. Usually a blue or yellow keel or paint stick is satisfactory for marking piling. On greasy, treated timber piling, a pressure can of paint generally gives the best results.

Pile Hammers: Pile hammers are normally of the following types:

A. Gravity or Single Acting Steam/Air Hammer: Employs a cable or steam or air to raise the hammer and has a driving energy equal to the weight of the ram multiplied by the height of the fall.

B. Double Acting Steam/Air Hammer: Employs steam (or compressed air) to raise the hammer and to accelerate the hammer on its downward fall.

C. Diesel Hammer, Open End: Returns the hammer by an explosion of diesel fuel at the time of impact, which increases the impact or driving energy furnished to the pile.

D. Diesel Hammer, Closed End: In the upward flight of the ram, compresses air to form an "air spring" to force the ram downward.

Before using a pile hammer it should be identified by name and size. The necessary data on ram weight, fall or length of stroke or bounce chamber pressure for closed end diesel hammers, hammer cushion and pile cushion (for concrete pile) must be obtained and checked against the information on the Form ITD-969 (Exhibit 505-1) previously submitted for the wave equation analysis by the contractor. Change in the hammer system may require a new wave equation analysis be run. It is particularly important to observe the condition, size and material of the hammer cushion. A compressed, worn or damaged hammer cushion should be replaced.

During driving, the hammer should be checked for proper operation. Length of stroke should be checked for single acting air/steam hammer and proper air/steam pressure should be obtained for double acting air/steam hammers. Since the resistance to pile penetration determines the height of return for the ram on diesel hammers, the stroke or delivered energy will increase with increasing resistance. The rated energy will not be developed until the hammer is at full stroke. Therefore, the driving energy cannot be accurately be determined where large penetrations per blow occur.

Stroke on open-ended diesel hammers is directly proportional to the blows/minute delivered by the hammer. Stroke heights can be calculated from the blows/minute using a Saximeter which records the blows acoustically and then calculates stroke from the blows/minute. Contact the Geotechnical Engineer if you need a Saximeter.

Closed-end diesel hammers show increasing bounce chamber pressure with increased energy. Monitoring bounce pressure is the only way of determining the energy developed. A Saximeter may still be used for counting blows but will not be of value in determining driving energy. Bounce pressure may require correction for altitude and hose length.

Safe driving operations require that equipment be in good condition and that workmen are safety-conscious. Cables, connections, and safe handling of the piling should be observed. The inspector must be especially careful when making measurements in the area around driving operations. The operator and workmen should be made aware of his presence at all times.

Ordering Piling

The Standard Specifications require the contractor to order piling after the engineer has authorized the length of piling to be ordered.

If test piles are set up on the project, the authorized pile lengths should be based on the test pile data. If test piles are not set up, the engineer should base the authorized length of piling on the pile lengths indicated on the plans.

Pile lengths should be based on the test pile results or the lengths shown on the plans and not on the lengths the contractor prefers to order. Standard pile lengths are 40, 50 or 60 feet. When ordering piles, consideration should be given to minimizing the splices and cutoff and utilizing all the piling.

Should the contractor choose to order the pile before the test piles are driven, he should be informed in writing that he is assuming full responsibility for the length and quantity.

Determining Pile Bearing Capacity

The intent is to use the wave equation analysis for determining pile bearing. Necessary data on pile driving equipment (Exhibit 505-1) must be submitted to the Geotechnical Engineer at the Central Materials Laboratory two weeks in advance of pile driving so that the wave equation analysis can be performed. The wave equation analysis is used to determine the hammer driving capacity and prepare the pile driving criteria, stroke, penetration resistance, bounce pressure, driving stresses, etc. If the proposed hammer is judged not adequate for the job, the contractor shall be required to use a different hammer. The contractor should be requested to provide this information at the preconstruction conference. If you are unable to obtain wave equation analysis results, the dynamic formula in the Standard Specifications may generally be used.

Wave equation analysis results are forwarded to the Districts by the Central Materials Laboratory. Results include the following:

A. Graph showing allowable bearing versus blow count per last foot and/or inch of driving (see Exhibit 505-6). This graph will generally show curves for different hammer stroke heights for open-end or bounce pressure for closed-end diesel hammers. Graphs may be sent for various pile lengths since the wave equation is based on definite pile lengths for bearing computations. Use the graph which corresponds to piling length being driven.

B. Graph showing stresses induced in piles during driving versus blow counts. This graph can be used to determine the maximum blow counts which should not be exceeded in order to prevent pile damage (Exhibit 505-6).

   The penetration resistance for refusal will be shown in the cover letter, Exhibit 505-6, accompanying the bearing capacity graphs. Pile refusal is established to minimize potential damages, particularly where piles are driven into rock.

The inspector shall count the number of blows at the last foot or inch of driving (or measure pile penetration in last ten blows) and apply it to the graph to determine the allowable pile bearing capacity. For single acting steam/air or open-end diesel hammers, the stroke height of the ram shall be observed in order to use the proper curve in the graph.

The design pile bearing must be achieved in at least two consecutive feet of penetration before driving can be stopped. When piles are driven to refusal or in very hard or dense materials, the blow counts could be very high and in these cases, blow counts per inch instead of per foot can be used to determine pile bearing. If the wave equation analysis indicated that a pile may be overstressed and damaged at a certain blow count and stroke, this level of driving resistance must never be exceeded.

In driving battered piles, the stroke height must be adjusted to compensate for the increased ram friction and inclination. A stroke height adjustment table is generally attached to the letter transmitting the pile driving criteria or can be found in the operating manual of the Saximeter II.

Notify the Geotechnical Engineer in the Materials Section as soon as possible when any driven pile achieves design bearing at a penetration of less than 50% or more than 150% of the estimated pile penetration shown in the plans so adjustments can be made in the pile driving criteria if necessary.

Pile Stresses

Any driven pile must remain structurally intact and not be stressed to its structural limits during both its service life (static capacity) and during driving (dynamic capacity). This requires that limits be placed on (1) maximum allowable design stresses during the service life, and (2) maximum allowable driving stresses during installation (temporary). In most cases, the highest stress levels occur in a pile during driving.

The ultimate static resistance specified is usually 2.0 or 2.5 times the design load. In many instances, pile damage occurs because of excessive stress levels generated in the pile during driving.

Two methods available for determining driving stresses are: (1) wave equation analysis, and (2) use of dynamic pile analyzer during pile driving. Ideally, the maximum allowable driving stress value permitted by the specifications should be based on the accuracy and reliability of the method used for determining the actual driving stresses. This would require two sets of recommendations: (1) for projects where design and construction control is only by wave equation analysis (lower maximum allowable driving stress values), and (2) for projects where wave equation analysis is used for design and the dynamic pile analyzer for construction control (higher maximum allowable driving stresses). In order to keep the recommendations simple and clear, only one set of recommendations is provided assuming that only wave equation analysis is used for both the design and construction control.

The following discussion of allowable pile driving stresses is grouped into four categories:

A. <u>Steel H, Steel Pipe (Top Driven), and Steel Monotube Piles</u>: Published literature generally recommends limits on driving stresses (tension and compression) in the range of 0.85 Fy to 1.1 Fy. (Fy is the minimum yield strength of steel.) The reasons for higher allowable stress than the yield strength of steel are (1) temporary duration of the load, (2) the strain of hardening of steel under repeated hammer impact, and (3) a recognition of the fact that usually the steel members have a higher strength that the specified minimum yield strength (Fy).

B. <u>Conventionally Reinforced Concrete Piles</u>: High stress or strain rates lead to increased compressive strength in concrete. This factor is offset by the fatigue effects under repeated high level stress cycles.

   Concrete is weak in resisting tensile stresses and conventionally reinforced concrete piles are frequently damaged because of high tensile stresses generated during easy driving situations.

C. <u>Prestressed Concrete Piles</u>: The effective prestress in concrete piles must be accounted for in computing net compressive or tensile stresses. Usually these piles are required to have at least 4800 kPa (700 psi) effective prestress (AASHTO 1983).

D. Timber Piles: Timber is known for its ability to withstand transient loads, such as pile driving. This fact must be tempered somewhat by the fatigue properties.

Recommendations

A. The recommended maximum allowable driving stresses for steel H, steel pipe (top driven), and steel monotubes are as follows:

Maximum tensile and compressive driving stress = 0.9 Fy

A. The recommended maximum allowable driving stresses for conventionally reinforced concrete piles are as follows:

Maximum compressive driving stress = 0.85 /fNc

Maximum tensile driving stress = 3 /fNc

C. The recommended maximum allowable driving stresses for prestressed concrete piles are as follows:

Maximum net compressive driving stress = 0.85 /f Nc B effective prestress

Maximum net tensile driving stress = 3 /fNc + effective prestress

D. The recommended maximum allowable driving stresses for timber piles are as follows:

Maximum compressive or tensile driving stress = 3 x (maximum allowable compressive or tensile design stress recommended by AASHTO 1983).

E. It is recommended that a wave equation analysis be performed during the design stage to estimate pile driving stresses and to make sure that the pile cross-section is sufficient for obtaining necessary embedment without exceeding the maximum allowable driving stress limits recommended in (A) through (D).

In easy driving situations (small resistance to pile driving), high tensile stresses are generated; therefore, make sure that the tensile driving stress limitation is not exceeded. Typical easy driving situations are (1) at the beginning of driving, and (2) when the pile penetrates into soft or loose soil layers which offer little resistance to the pile tip.

In hard driving situations (high resistance to pile driving), high compressive stresses are generated; therefore, make sure that the compressive driving stress limitation is not exceeded. Typical hard driving situations are (1) at the end of driving for end-bearing piles, and (2) when the pile penetrates into very dense soil layers.

F. It is recommended that the wave equation analysis and/or, preferably, the dynamic pile analyzer be used for construction control of the pile driving. The wave equation analysis will provide an estimate of the driving stresses generated, whereas the dynamic pile analyzer will measure the driving stresses for each hammer blow during driving.

G. If pile driving becomes difficult and refusal is met before required tip elevation is obtained, pile driving should be stopped; otherwise, piling may be damaged.

V - Pile Splices

Refer to the plans and specifications to determine how splices are to be made. Generally, splices are not paid for if they are for the contractor's convenience; however, it may be practical and economical to pay for splices when cutoffs become excessive and the payment for cutoffs exceeds the cost of a splice.

Piling lengths exceeding 18 m (60 ft.) will usually require a splice. Depending on piling quantities authorized to be ordered; the depth driven, cutoff lengths, etc., will be factors in determining if any splices will be paid for each pile.

In summary, an analysis should be made to determine splicing cost versus cutoff lengths. The decision to splice or waste cutoffs should be based on this analysis.

VI - Documentation for Pay Quantity

The ITD-792, Summary Report of Pile Driving (Exhibit 505-4), shall be the source document for pay quantity. Form ITD-970, Test Pile Record (Exhibit 505-2), is used for test piles. The diary shall be used to verify the activity, date, and location of the work.

Piling shall be measured and reported to the nearest 0.3 meter (ft.).

VII - Reports

The following reports are required for pile driving:

A. Form ITD-970, Test Pile Record, Exhibit 501-2

If a test pile is set up, the intent of this form is to present a log of the bearing value of the pile throughout its depth of penetration. Sufficient readings should be taken to permit the plotting of a graph of bearing versus penetration.

A separate report is required for each test pile. Handwritten reports, neatly prepared, are acceptable; and recopying is discouraged. Enough information should be furnished about the hammer being used to permit subsequent recalculation of the bearing values.

B. Form ITD-971, Individual Pile Driving Record, Exhibit 505-3

This form is used to record data for each production pile driven.

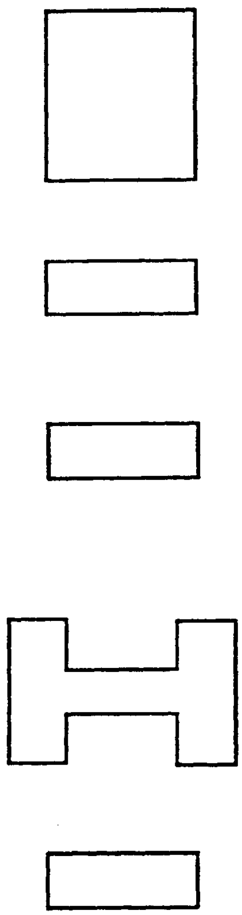
C. Form ITD-972, Summary Report of Pile Driving, Exhibit 505-4

The summary of pile driving is shown on this report.

D. Form ITD-973, Summary Report of Pile Driving (Pile Layout), Exhibit 505-5

This form shows the locations of driven piles along the centerline of the structure.

PILE DRIVING HAMMER DATA NEEDED FOR WAVE EQUATION ANALYSIS



HAMMER

MANUFACTURER * ______________________ MODEL * ______________

TYPE * ____________________ RATED ENERGY ______________ (Ft.-Lb.)

RAM WEIGHT ______________ (Lbs.) MAX. STROKE ____________ (Ft.)

STRIKE PLATE

WEIGHT * _________________ (Kips) THICKNESS ______________ (In.)

CROSS SECTION AREA ________________________________ (Sq. In.)

HAMMER CUSHION

MATERIAL * ______________________ THICKNESS * ____________ (In.)

CROSS SECTION AREA * ______________________________ (Sq. In.)

ELASTIC MODULUS ______________________________________ (Ksi)

COEFFICIENT OF RESTITUTION __________________________________

HELMET (INCLUDING ADAPTOR)

WEIGHT * ____ (Kips)

PILE CUSHION (FOR CONCRETE PILES ONLY)

MATERIAL * ____________________ THICKNESS * ________________ (In.)

AREA * _______________ (Sq. In.) ELASTIC MOD. ______________ (Ksi)

COEFFICIENT OF RESTITUTION __________________________________

* ESSENTIAL DATA, MUST BE PROVIDED.

DATA SUBMITTED BY ____

COMPANY ____

PHONE ____

DATE ____

D-970 2-90

TEST PILE RECORD

PROJECT NO. ______________________________

PROJECT NAME ______________________________

STRUCTURE ______________ LOCATION ______________

PILE PEN. (FT.)	VERT. HAMMER STROKE (FT.)	BLOW PER FOOT	BLOW PER INCH	ALLOW-ABLE PILE CAP. (TON)

HAMMER ______________________________

RAM WEIGHT ______________ (LBS.)

RATED ENERGY ______________ (FT.-LBS.)

PILE TYPE ______________ SIZE ______________

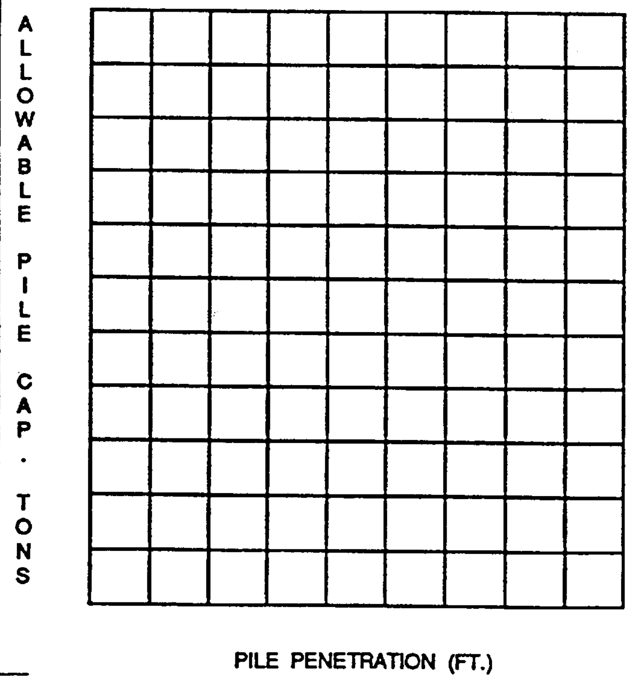
PILE LENGTH ______________________________

TIP PROTECTOR ______________________________

PILE LOCATION ______________________________

GROUND LINE ELEV.(FT.) ______________

PILE DESIGN LOAD ______________ (TONS)



REMARKS ______________________________

____

____

INSPECTOR ______________

DATE ______________

DISTRIBUTION:

RESIDENT ENGR. (ORIGINAL) ☐

MATERIALS (GEOTECH. ENGR.) ☐

ITD-971 2-90

INDIVIDUAL PILE DRIVING RECORD

PROJECT NO. ______

PROJECT NAME ______

PILE LOCATION: ABUT. NO. ______ PIER NO. ______ PILE NO. ______

PILE TYPE ______ SIZE ______ TIP PROTECTOR ______

PILE LENGTH ______ (FT.) CUT OFF ______ (FT.) DESIGN LOAD ______ TONS

PILE CUT OFF ELEV. ______ PILE TIP ELEV. ______

HAMMER TYPE ______ MANUFACTURER ______ MODEL ______

MAXIMUM ENERGY ______ (FT. - LB.) WEATHER ______

PILE PEN. (FT. - IN.)	HAMMER STROKE (FT.)	ALLOW. PILE CAPACITY (TON)	PILE PEN. (FT. - IN.)	HAMMER STROKE (FT.)	ALLOW. PILE CAPACITY (TON)	PILE PEN. (FT. - IN.)	HAMMER STROKE (FT.)	ALLOW. PILE CAPACITY (TON)

REMARKS ______

______

______

D. ______ Inspector ______

DISTRIBUTION: ☐ Resident Engineer (ORIGINAL) ☐ Materials (Geotechnical Engineer)

ITD-972 2-90

SUMMARY REPORT OF PILE DRIVING

(PILE LENGTH AND CAPACITY)

PROJECT NO. ____________________

PROJECT NAME ____________________

STRUCTURE ____________________ LOCATION ____________________

PILE TYPE ____________ SIZE ____________ TIP PROTECTOR ____________

HAMMER TYPE ____________ MANUFACTURER ____________ MODEL ____________

RAM WEIGHT ____________ (LBS.) MAXIMUM STROKE ____________ (FT.)

RATED ENERGY ____________________ (FT. - LB.)

ABUT. OR PIER NO.	PILE NO.	DATE DRIVEN	PILE LENGTH (FT.)	CUT - OFF (FT.)	NET LENGTH (FT.)	VERT. HAMMER STROKE (FT.)	BLOW PER FT. OR IN.	ALLOW-ABLE BEARING CAPACITY (TON)	DESIGN LOAD (TON)

REMARKS ____________________

____

____

DATE ____________________

DISTRIBUTION:

- ☐ RESIDENT ENGINEER (ORIGINAL)
- ☐ CONTRACT ADMINISTRATION
- ☐ BRIDGE
- ☐ MATERIALS (GEOTECHNICAL ENGINEER)
- ☐ DISTRICT_____ MATERIALS ENGINEER
- ☐ CENTRAL FILES

ITD-973 2-90

SUMMARY REPORT OF PILE DRIVING

(PILE LAYOUT)

PROJECT NO. ____________________

PROJECT NAME ____________________

STRUCTURE ____________________ LOCATION ____________________

PILE TYPE ____________ SIZE ____________ TIP PROTECTOR ____________

HAMMER TYPE ____________ MANUFACTURER ____________ MODEL ____________

RAM WEIGHT ____________ (LBS.) MAXIMUM STROKE ____________ (FT.)

RATED ENERGY ____________________ (FT. - LB.)

PILE LAYOUT

REMARKS ____________________

D\ ____________________

DISTRIBUTION:

☐ RESIDENT ENGINER (ORIGINAL)

☐ CONTRACT ADMIN.

☐ BRIDGE

☐ MATERIALS (GEOTECH. ENGR.)

☐ DIST.____ MTLS. ENGR.

☐ CENTRAL FILES

ITD 500 8/84

STATE OF IDAHO
TRANSPORTATION DEPARTMENT

Intra-Department Correspondence

DATE: SEPTEMBER 2, 1988

PROJECT No:
KEY No: I-90-1(75)61

WALLACE VIADUCT
SHOSHONE COUNTY

TO: DISTRICT 1 RESIDENT ENGINEER

FROM: MATERIALS SECTION
- GEOTECHNICAL ENGINEER

SUBJECT: PILE DRIVING CRITERIA

Transmitted herein are the pile driving criteria developed by wave equation analyses for Wallace viaduct per your request.

It was assumed that diesel hammer Kobe K-35 will be used to drive piles HP 14 x 117 (with points APF 77600) through 10 to 30' of very dense sand and gravel to end bearing in argillite bedrock.

For piles to achieve the design load of 135 tons/pile, the following blow counts are recommended:

Hammer's Stroke, ft.	Blows/foot	Blows/inch
5.0	85	7
5.5	68	6
6.0	52	5
6.5	45	4
7.0	36	3
7.5	32	2.7
8.0	26	2
8.5 or higher	22	2

Practical pile refusals are as follows:

75 blows/foot @ 8' stroke or higher (or 6 blows/inch)
90 blows/foot @ 7' stroke (or 7 blows/inch)
100 blows/foot @ 6' stroke (or 8 blows/inch)

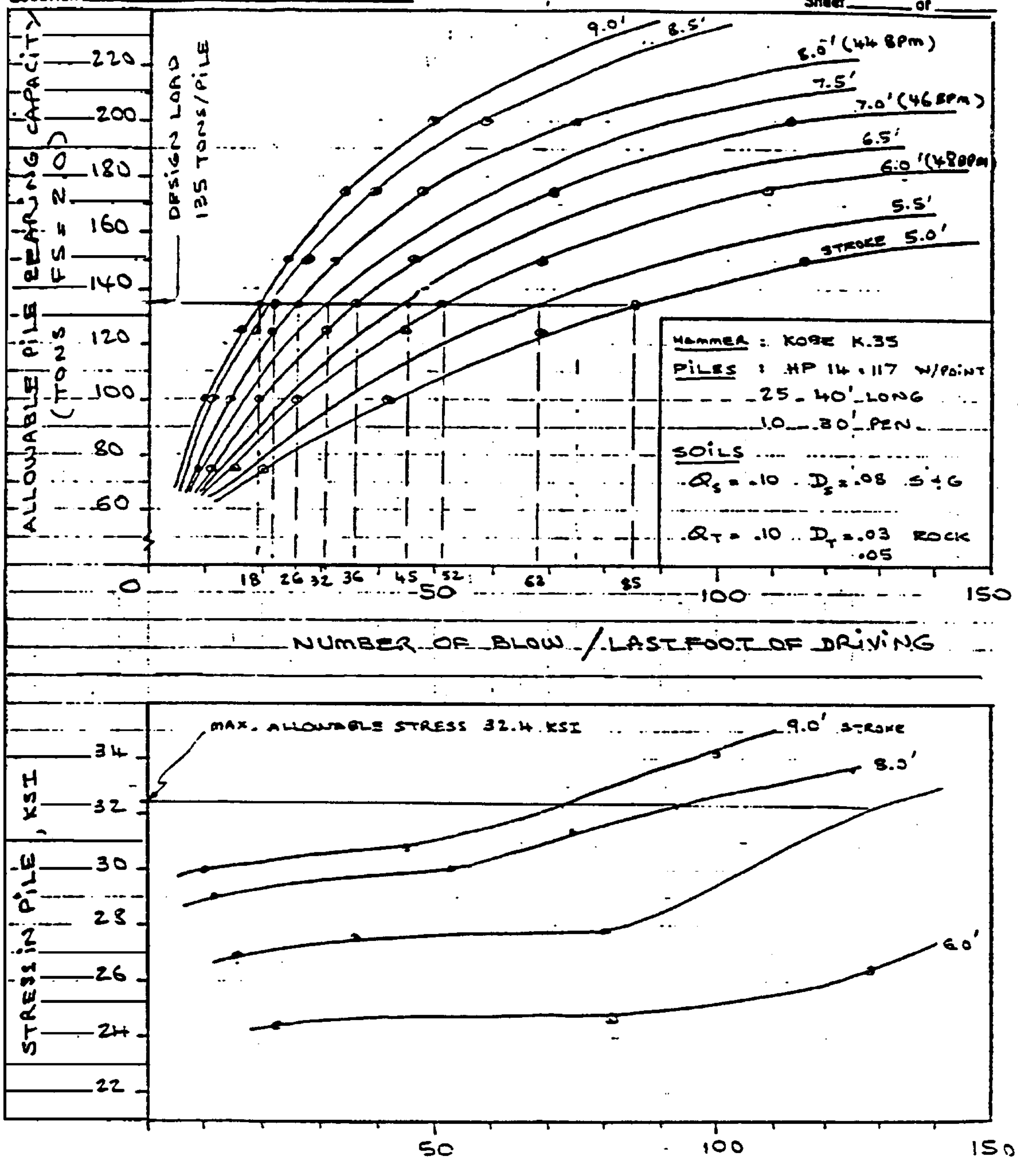
To avoid potential pile damages, the hammer stroke should be kept under 8.5' by adjusting the fuel setting if necessary and blow counts should never exceed:

7 blows/inch if stroke height is 8' or higher
10 blows/inch if stroke height is 7'
13 blows/inch if stroke height is 6'

OH-404 11-74

STANDARD COMPUTATION SHEET

Description Wave Equation Results
Location Wallace Viaduct
Sheet ____ of ____



Computed by ____ Date 9 1 88 Project No. ID 90 (75) 51
Checked by ____ Date ____ File or Book No. ____
Designed ____ Checked ____

District 1 Resident Engineer
September 2, 1988
Page 2

The Saximeter II should be used to determine hammer stroke heights and blow counts. For battered piles, use Figure 2 or Table 1 in the "Operating Procedure for Saximeter II" to correct the stroke heights before using the above criteria.

The attached graphs show the results of the wave equation analyses.

If you have any questions, please call.

MTLTRI1:TB:db

cc: Matls - R. Smith w/att
Bridge
CAS w/att
Dist 1 Engr
Dist 1 Matls w/att
Geot Engr w/att

I-20
Sheet 1
Nov. 89

LIST OF APPROVED POINTS FOR STEEL H PILES
(Prepared by Materials Section)

POINT TYPE - USE	APF	DFP(1)	ICE(2)	VERSABITE(3)
General Use- Long Web Support	75000 (35#)(4)			
Square Tip- Hard Drive Friction- End Bearing in Gravels-Level Rock	75500 (75#)			VB-300H (74#)
General Use- Wide Contact Area	75600 (50#)			
General Use- Most Common Gravel Point	75750 (28#)	H-777	HPH Series (27#)	VB-300 (25#)
Most Commonly Used- Rock Point	77600 (31#)	H-776	HPH-RB Series	VB-300P (33#)
Slim Section- Aid Penetration Through Compressive Strata Over Rock (5)	77750 (29#)	H-777		
Hard Face-Continuous Cutting Edge For Hard Driving	77675			
Similar To 77600 With Hard Face Cutting Teeth	77600S			
Rock Injector Point- Not Commonly Used Now	80500	H-805	HPH-RB Series (38#)	

APF - Associated Pile & Fitting
DFP - Dougherty Foundation Products
ICE - International Construction Equipment
VERSABITE - Versabite Foundation Accessories

NOTES

(1) Uses ASTM A148 Castings 90/60
(2) Minimum Web Support
(3) Uses ASTM A148 Castings, Accepted By Washington & Oregon, But We Need Dimensions To Confirm Equivalency
(4) (35#)- Weight Of Point For 12 Inch H Piles
(5) To Maximum Friction In Bearing Zone- Not Recommended For Boulders.

I-20
Sheet 2
Nov. 89

LIST OF APPROVED POINTS, SHOES AND BOOTS FOR PIPE PILES

(Prepared by Materials Section - ITD)

POINT, SHOE, BOOT TYPE	APF	DFP	ICE	VERSA BITE
60 Deg. Conical Point Inside Fit	P 13006	P-77R	60 HD Series	VB-900 Series
60 Deg. Conical Point Outside Fit	P 13000			VB-600 Series
Open-End Cutting Shoe Inside Fit	O 14001	DFP 0140	ICE Inside Cutting Shoe	VB-700 Series
Closure Boot	PS-17000	PB-170	ICE Round Tite Boot	

NOTES:

APF - Associated Pile & Fitting Corp., PO Box 1043, Clifton , New Jersey 07014

DFP - Dougherty Foundation Products, Inc., PO Box 688, Franklin Lakes, New Jersey 07417

ICE - International Construction Equipment, Inc., 301 Warehouse Drive, Matthews, N. Carolina 28105

VERSABITE - Versabite Foundation Accessories, 3475 Gribble Road, Matthews, N. Carolina 28105

I-20
Sheet 3
Nov. 89

LIST OF APPROVED SPLICERS FOR STEEL PILES

(Prepared by Materials Section - ITD)

PILE TYPE	APF	DFP	ICE	VERSA BITE	REMARKS
PIPE PILES	S-18000	S-1800	ROUND TITE COUPLER	VB 800 Series	(1)
H PILES	HP-30000	HP-300	HSA Series	VS Series	

NOTES:

APF - Associated Pile & Fitting Corp., PO Box 1043, Clifton, New Jersey 07014

DFP - Dougherty Foundation Products, Inc., PO Box 688, Franklin Lakes, New Jersey 07417

ICE - International Construction Equipment, Inc., 301 Warehouse Dr., Matthews, N. Carolina 28105

VERSABITE - Versabite Foundation Accessories, 3475 Gribble Road, Matthews, N. Carolina 28105

(1) Pre-fabricated splicers for pipe piles are usually not welded to piles unless the piles are designed for tensile forces. When splicer is not fillet welded to pile,it will not provide full strength in bending.

PRESTRESSING CONCRETE (506)

I - General

The Bridge Section will approve working plans which will show the type of prestressing, the bed layouts, calculations for elongation, friction losses, sequences for stressing and detensioning, etc. The Prestressed Concrete Institute's Manual for Quality Control for Plants and Production of Precast Prestressed Concrete Product is incorporated into the Standard Specifications and will be adhered to on tolerances for the fabrication of precast, prestressed concrete items.

II - Prestressing Methods

Prestressing may be accomplished by pretensioning, post-tensioning, or a combination of both methods.

A. Pretensioning: The stressing tendons are tensioned before the concrete is placed. After the concrete has developed a specified strength, the anchorages for the tendons are released and the forces in the tendons are transferred to the concrete.

There are two systems of pretensioning the tendons. One system is the tensioning of each strand individually. This is referred to as single strand tensioning. The other system consists of pulling two or more strands simultaneously. This is referred to as multiple strand tensioning.

B. Post-Tensioning: The stressing tendons are installed in voids or ducts within the concrete and are stressed and anchored after the concrete has developed a specified strength. As a final operation, the voids or ducts are pressure grouted.

C. Combination Method: Some of the stressing tendons are pretensioned and some are post-tensioned. In this method, requirements for pretensioning and post-tensioning apply to the respective stressing elements used.

See Sections 502 and 503 for inspection of the concrete and metal reinforcement.

Special attention should be given the location of the reinforcing bars extending out of precast members. Bars which are to be used as connecting bars between precast and poured-in-place construction should be extended from the precast member the distance and at the exact location shown on the plans. Mortar coatings adhering to bars protruding beyond the surface of precast members must be removed. Bending of these bars to facilitate moving or handling must be kept to an absolute minimum.

In the event that stressing is not done in a continuous operation, members should not be handled before they are sufficiently stressed to sustain all forces and bending moments due to handling.

When handling beams, it is imperative that they be maintained in an upright position at all times and picked up and supported only near the ends. Disregard of handling requirements may lead to damage or collapse of the member.

It is difficult and impractical to establish fixed criteria regarding the acceptability of prestressed concrete members with respect to appearance. All members should be fabricated in a workmanlike manner without cracks or other defects and true to the dimensions shown on the plans. However, it is recognized that certain cracks and surface defects may not be detrimental from the standpoint of structural integrity and may be harmless if remedied by proper repair. The Construction Section should be consulted when cracks or other defects are found as to the acceptability of the member or method of repairing the defect.

Concrete of the consistency required for prestressed concrete is sometimes difficult to place without surface flaws. This is especially true of irregular shapes, such as "I" beams which have sloping surfaces that tend to trap air and cause surface imperfections termed "bug holes." "Bug holes" have no significance structurally and are of concern only from an appearance standpoint.

If the concrete has been steam cured, it will be whiter in appearance than concrete made with the same materials which is not steam cured. Substitution of from one-third to one-half of the cement with white portland cement in mixes made for patching and hand rubbing is common practice.

Each member should be marked in accordance with the erection drawing for identification.

Each District Materials Section has a condensed copy of California's Prestress Manual. There is some very good information in this manual and everyone should review it prior to stressing operations. It should be noted that this manual is a guide, not a specification.

III - Prestressing Guidelines

It is recommended the following procedures be used prior to and during all prestressing operations:

A. No tendons should be stressed until elongation calculations have been adjusted for the materials being used.

In order to accomplish this, the correct value of the modulus of elasticity must be obtained for the strands being used.

B. On any prestress operation, an electrohydraulic type load cell (Model HWI-B Pressure Transducer) and qualified operator should be on the job to monitor enough tendons to assure the provided calibration curve is not in error. (This particular piece of equipment was used in determining the reason for the poor correlation in the above-mentioned problem.)

C. All jacks used for prestressing purposes should be calibrated within one year of its use. Headquarters' Materials Section will have, in the near future, facilities to calibrate any prestressing jack up to 4448 kN (500 tons).

D. All equipment necessary to check the minimum efflux time of the grout should be on the job site in advance of the post-tensioning operation.

IV - Safety

Special safety precautions are required when working around prestressed stringers because of their size and the stress contained in them and the auxiliary equipment. Some of these are:

A. Stay away from prestressing strands when stress is being applied.

B. Do not stand, reach or walk under stringers which are being supported by a crane.

C. Stand clear when stringers are being raised, as they may swing when coming off the ground.

V - Documentation for Pay Quantity

See Section 502, Number IX.

VI - Reports

See Section 502, Number X.

BEARING PADS AND PLATES (507)

I - General

Bearing units serve two primary functions on a structure, they transfer the loads uniformly from the superstructure to the substructure and allow for movement of the superstructure due to thermal expansion or contraction and deflections due to live loads on the structure.

The bearing units are generally one of the following two types:

- TFE Bearings: A top plate made of steel with Teflon sliding on polished stainless steel. An elastomeric pad is attached underneath the top plate.
- Elastomeric Bearings: Solid or laminated neoprene pads.

The type of bearing specified for any individual structure depends upon the load and movement that will be expected to occur.

The importance of providing proper bearings cannot be overemphasized. Unless bearing surfaces are produced that will come in contact completely with the bearing supports, the structure may develop serious structural weaknesses.

The bearing area for a member bearing on elastomeric bearing pads must be finished to a true plane. This bearing area must be constructed so as to give uniform bearing on the entire area. These bearing areas must be formed with unyielding supports when the members are cast.

A manufacturer's certificate is required and this document should be on file with the Division prior to installation. A visual inspection of the pads should also be made by the Inspector prior to installation.

Steel plates must meet the requirements of Structural Metals. The shop drawings will be approved by the Engineer prior to fabrication of the units.

II - Documentation for Pay Quantity

The diary shall be used to verify the activity, date, location of the work, and all dimensional checks.

There is no item for pay, as the cost is to be included in other contract items.

III - Reports

None.

CORRUGATED PLATE PIPE (508)

I - General

See Section 601 for surveying and inspection requirements. In addition, an itemized statement of the size and number of plates in each shipment, and detailed fabrication and erection plan for each pipe is also required.

Bedding and shaping of the foundation should conform to the requirements for conduits. A foundation with uniform bearing capacity is essential. Special bedding or backfilling materials or procedures will be outlined in the special provisions, if required. Compaction of backfill material under the haunches should be given close attention.

High strength steel bolts are required by the specifications to be used in assembling the plates. Bolt heads are marked with three radial lines 120E apart.

Bolts shall be tightened in accordance with the manufacturer's recommendations.

II - Documentation for Pay Quantities

Complete field notes will be required for the computation of structural excavation and compacting backfill. The length should be measured in the field and the pay quantity entered on diary or pipe sheet, if being used. The diary shall be used to verify the activity date and location of the work. Corrugated plate pipe will be measured and reported to the nearest 0.3 meter (ft.).

III - Reports

None.

DECK SEAL CONCRETE B CLASS CH (509)

DELETED

DECK SEAL CONCRETE B LATEX MODIFIED CONCRETE (510)

TO BE PUBLISHED AS SOON AS POSSIBLE

ASPHALT WATERPROOF MEMBRANE (511)

TO BE PUBLISHED AS SOON AS POSSIBLE